Technical Report CHL-99-13
July 1999

US Army Corps of Engineers
Waterways Experiment Station

Monmouth Beach, New Jersey: Beach-Fill "Hot Spot" Erosion Evaluation

Report 2
Functional Design of Shore-Protection Alternatives for Beach-Fill Longevity

by S. Jarrell Smith, Gregory L. Williams, Nicholas C. Kraus

Approved For Public Release; Distribution Is Unlimited

Prepared for U.S. Army Engineer District, New York

Technical Report CHL-99-13
July 1999

Monmouth Beach, New Jersey: Beach-Fill "Hot Spot" Erosion Evaluation

Report 2
Functional Design of Shore-Protection Alternatives for Beach-Fill Longevity

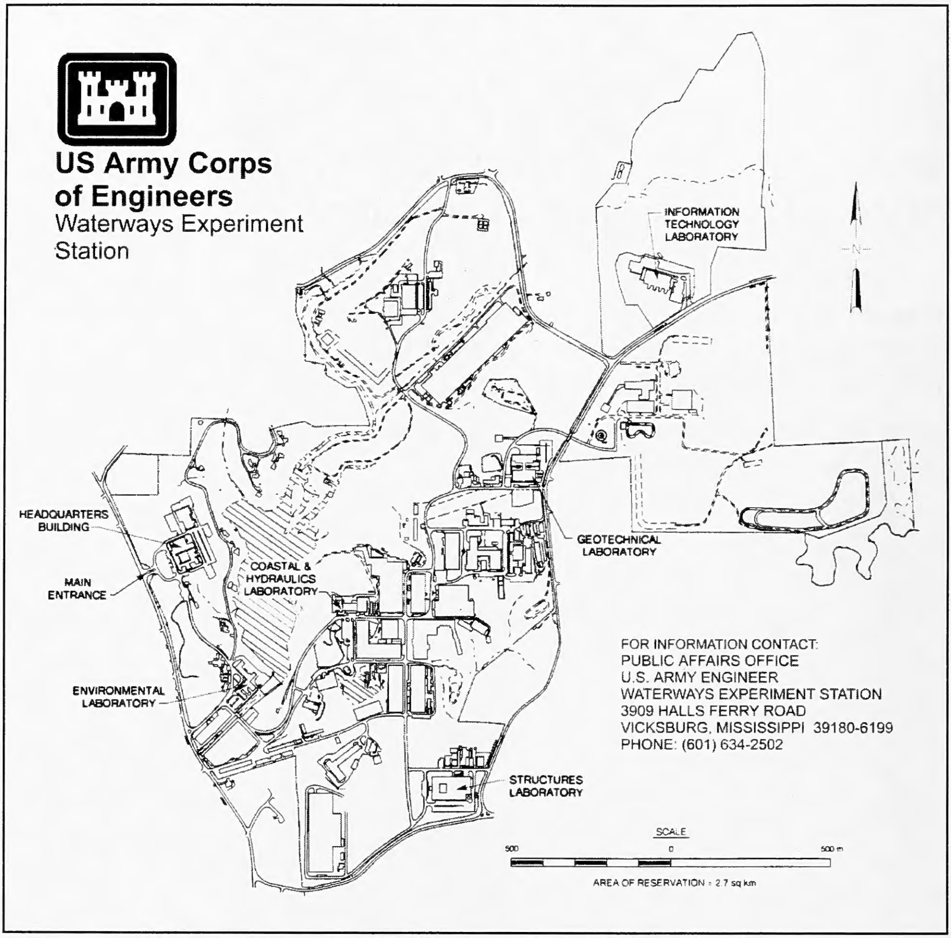
by S. Jarrell Smith, Gregory L. Williams, Nicholas C. Kraus

U.S. Army Corps of Engineers
Waterways Experiment Station
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

Report 2 of a series

Approved for public release; distribution is unlimited

Prepared for U.S. Army Engineer District, New York
New York, NY 10278-0090



Waterways Experiment Station Cataloging-in-Publication Data

Smith, S. Jarrell.
Monmouth Beach, New Jersey : beach-fill "hot spot" erosion evaluation. Report 2, Functional design of shore-protection alternatives for beach-fill longevity / by S. Jarrell Smith, Gregory L. Williams, Nicholas C. Kraus ; prepared for U.S. Army Engineer District, New York.
77 p. : ill. ; 28 cm. — (Technical report ; CHL-99-13 rept.2)
Includes bibliographic references.
Report 2 of a series.
1. Beach erosion — New Jersey. 2. Shore protection — New Jersey. 3. Coastal zone management — New Jersey. 4. Monmouth Beach (N.J.) I. Williams, Gregory L. II. Kraus, Nicholas C. III. United States. Army. Corps of Engineers. New York District. IV. U.S. Army Engineer Waterways Experiment Station. V. Coastal and Hydraulics Laboratory (U.S. Army Engineer Waterways Experiment Station) VI. Title. VII. Title: Functional design of shore-protection alternatives for beach-fill longevity. VIII. Series: Technical report (U.S. Army Engineer Waterways Experiment Station) ; CHL-99-13 rept.2.
TA7 W34 no.CHL-99-13 rept.2

Contents

List of Figures

List of Tables

Preface

In March 1998, the U.S. Army Engineer District, New York (New York District), requested that the U.S. Army Engineer Waterways Experiment Station (WES), Coastal and Hydraulics Laboratory (CHL) provide a functional design to reduce the renourishment interval at a section of Monmouth Beach, New Jersey, that experiences a high rate of erosion. Ms. Lynn M. Bocamazo, Engineering Division, New York District, was the project study manager. WES is a complex of five laboratories of the U.S. Army Engineer Research and Development Center (ERDC).

This report documents the subject study, which developed and evaluated shore-protection alternatives and made estimates of the required stone sizes and volumes for the recommended alternative involving a coastal groin. The study encompassed an analysis of the functioning of groins at the site, analysis of the acting coastal processes, numerical simulations of shoreline change, and development of functional designs combined with coastal engineering analysis of the associated structures.

The study was performed by Mr. S. Jarrell Smith, Coastal Processes Branch (CPB), Coastal Sediments and Engineering Division (CSED), Mr. Gregory L. Williams, Coastal Evaluation and Design Branch, CSED, and Dr. Nicholas C. Kraus, CSED, all of CHL. Work was performed under the supervision of Mr. Thomas W. Richardson, Chief, CSED. Ms. J. Holley Messing, CPB, coordinated report preparation.

The Director and Assistant Director of CHL were Dr. James R. Houston and Mr. Charles C. Calhoun, Jr. (retired), respectively.

At the time of report publication, Commander of ERDC was COL Robin R. Cababa, EN.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
cubic yards	0.7645549	cubic meters
degrees (angle)	0.01745329	radians
feet	0.3048	meters
inches	2.54	centimeters
miles (U.S. nautical)	1.852	kilometers
miles (U.S. statute)	1.609347	kilometers
pounds (mass)	0.4535924	kilograms
square feet	0.0929	square meters
tons (2,000 pounds, mass)	907.1847	kilograms

1 Introduction

This chapter gives an orientation to the study site, problem statement, discussion of units of measurement employed in this report, and the organization of the report.

Overview of Study Site

The Borough of Monmouth Beach lies on the northern coast of New Jersey, approximately 5 miles[1] south of the entrance to Sandy Hook (Figure 1). Monmouth Beach is located in Section I of the Atlantic Coast of New Jersey Sandy Hook to Barnegat Inlet Beach Erosion Control Project, as described in the General Design Memorandum (GDM) prepared by the U.S. Army Engineer District, New York (New York District) (New York District 1989). An approximately 2,600-ft-long section of Monmouth Beach is eroding at a greater than anticipated rate. The seawall is becoming exposed, and the design objective of a 6-year nourishment interval is not being achieved.

The GDM (New York District 1989), Kraus et al. (1988), and Kraus, Gravens, and Mark (1988) review the geological structure and coastal processes acting along this coast with references made to the available literature. The barrier-spit shore along the Boroughs of Sea Bright and Monmouth Beach is heavily developed and fronted by seawalls and numerous types of groins. The width of the spit in this area varies from approximately 300 to 1,200 ft, and the elevation of the backland varies between approximately 3 and 12 ft referenced to mean low water (mlw). The median grain size of sediments on the nearshore beach profile on the north New Jersey shore was found to remain constant, at 0.33 mm from samples collected in 1953 and 0.30 mm for samples collected in 1985.

The longshore sand transport rate along the northern coast of New Jersey is greatly influenced by wave sheltering in the Atlantic Ocean because of the landmasses located to the north, notably, by Long Island, New York. As a result, Caldwell (1966) concluded that there was a regional divergent nodal area of transport located in the vicinity of Mantoloking, with predominant net transport directed to the north northward of this area and directed to the south southward of the area. With distance north of Mantoloking, the net longshore transport rate

---

[1] A table of factors for converting non-SI units of measure to SI units is found on page viii.

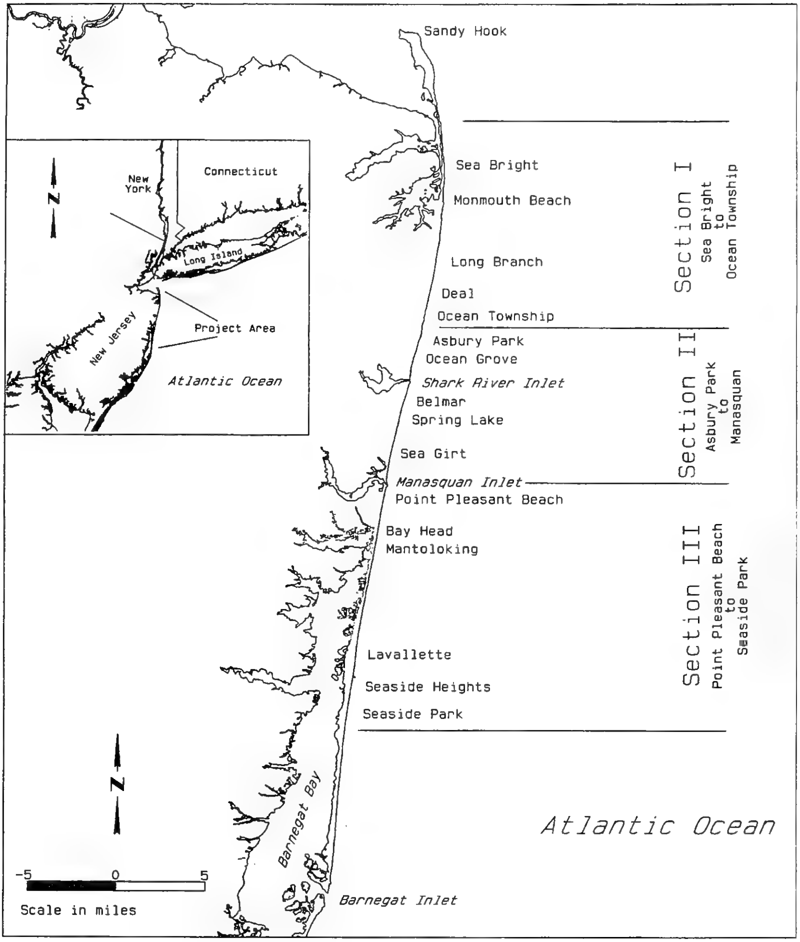


Figure 1. Site location map

increases because of increased sheltering. Transport rates deduced by Caldwell and validation of net direction of transport found in the present study are discussed in Chapter 3.

Problem Statement

Under Contract 1A of the Sea Bright to Ocean Township, New Jersey, Beach Erosion Control Project, fill was initially placed at the site in the autumn of 1994.

Between Groins 44 and 45, 519,500 cu yd of sand was placed as part of the initial beach fill. By October 1995, the area between Groins 44 and 45 (the hot spot) was severely eroded, and an emergency beach fill amounting to 290,200 cu yd of sand was placed. With the initiation of Contract 2 (placing beach fill to the south of Monmouth Beach) in June 1997, the erosion "hot spot" was again filled, requiring a volume of 564,600 cu yd of fill[1]. Including initial placement, the Monmouth Beach area between Groins 44 and 45 has been nourished three times for a total supplementary added amount of 1,374,000 cu yd. The subject area of beach and the locations of the two groins in relation to the New York District profile stationing is shown in Figure 2. Because the average renourishment interval for the project is 6 years, the Monmouth Beach area is experiencing fill loss at an unacceptably high rate.

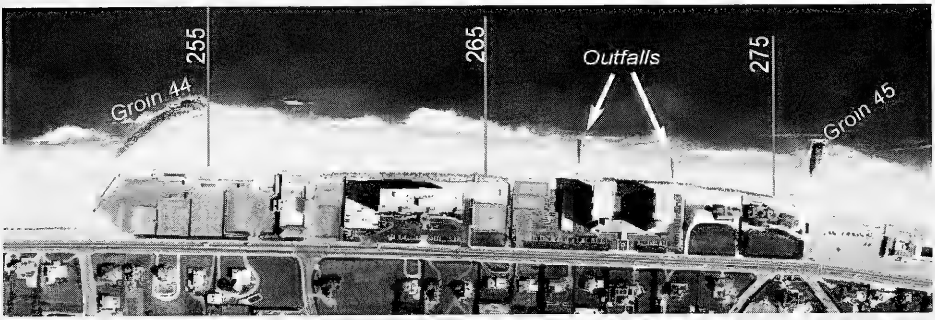


Figure 2. Locations of beach-profile surveys within hot spot (September 1996 photograph)

Among several possible causes considered, Smith, Gravens, and Smith (in preparation) concluded that the anomalously high loss rate along this 2,600-ft-long erosion hot spot is a consequence of its protrusion seaward with respect to the adjacent beaches. Thus, both (a) inefficiency of an originally short fill (high end-losses) and (b) partial isolation from adjacent beaches, especially the beach to the north (which prevents arrival of sand to the hot spot), create a situation where loss of material will be accelerated. The first reason (short fill) will be eliminated after fill material is placed to the south in continuation of the shore-protection project. Partial isolation of the area because of protrusion from the adjacent beaches will remain as a cause of a higher-than-average erosion rate.

To address the above issue of hot-spot erosion, in March 1998, the New York District authorized the present study by the U.S. Army Engineer Waterways Experiment Station (WES), Coastal and Hydraulics Laboratory (CHL). The objective of the study was to develop a functional shore-protection design for Monmouth Beach that would eliminate the need for intermediate remedial actions and bring the replenishment cycle to that of the full project (6 years).

---

[1] Personal Communication, 4 August 1998, Mr. Kenneth Malley, Construction Division, U.S. Army Engineer District, New York.

Structural alternatives in addition to fill were to be considered. The present study builds upon a preliminary study performed by Coastal Planning & Engineering, Inc. (CP&E) (1997), which developed a design involving construction of groins and placement of fill to increase the replenishment cycle to an estimated 3 years. The present study included a more comprehensive analysis of the local wave and shoreline data, as well as evaluation of a wider range of alternatives including T-head groins.

Units of Measurement

In this report, general discussion of wave data is made in SI (metric) units in conformance with standard oceanographic practice. However, because the main body of this report concerns an ongoing project for which engineering quantities have been and continue to be expressed in American Customary (non-SI) Units, such quantities are expressed in non-SI units. A table for conversion from non-SI units of measurement to SI (metric) units is presented on page viii.

Organization of this Report

This report is the second in a series on the topic of accelerated erosion rates of the beach fill at Monmouth Beach, New Jersey. The first report identifies factors contributing to the area of accelerated erosion at Monmouth Beach. The present report extends the conceptual recommendations of the first report by developing and analyzing shore-protection alternatives and estimating required stone sizes and volumes for the recommended alternatives.

This report consists of five chapters. Chapter 1 introduces the study site and objectives of the study. Chapter 2 gives a summary of the acting coastal processes and includes an analysis of directional wave data recently acquired in the vicinity of the site, information on tidal datums, and a description of the beach profile and sediments. Chapter 3 describes the procedures and results in development and evaluation of functional designs for increasing the renourishment interval. It covers an analysis for obtaining an empirical criterion for groin performance, numerical simulations of longshore sand transport and shoreline change, and selection of recommended alternatives. Chapter 4 contains the analysis procedure to develop construction details, in particular stone sizes and volume, for the groin extension identified in one of two recommended alternatives from Chapter 3. Chapter 4 also contains percent-damage curves for the groin extension. Chapter 5 gives conclusions and recommendations arrived at in this study.

Appendix A lists statistical summaries of the wave analysis for the Long Branch, New Jersey, directional wave gauge measurements.

2 Summary of Acting Coastal Processes

This chapter contains a summary of acting coastal processes at the Monmouth Beach erosion hot spot. An analysis of directional wave data collected at Long Branch, New Jersey is presented, as well as a description of beach sediments, information about vertical datums, and a description of beach-profile adjustment.

Long Branch Gauge Analysis

This section presents statistical information derived from directional measurements of waves at Long Branch, New Jersey. The Long Branch gauge (40.30° N, 73.97° W) is located approximately 2.7 km south of the hot spot at Monmouth Beach at a water depth of 10 m. A slope array, deployed by CHL for the New York District in 1994, was replaced with a directional wave gauge (DWG) in July 1995. Directional wave data are collected at a rate of 1 Hz for 1,024-sec-long bursts. Bursts of data are collected hourly during storm conditions and at 4-hr intervals during calmer wave conditions. Peak energy-based wave height (H_{mo}), period, and direction are determined and recorded for each data burst.

Wave periods and directions are not recorded for conditions in which the measured wave height is less than 0.2 m. Gaps in the wave record exist because of instrument failure and malfunction (approximately 23 percent of gauge deployment) with extended gaps in the data occurring between mid-November 1994 to mid-July 1995 and October 1996-February 1997. A summary of statistical parameters derived from the DWG measurements from January 1994 to March 1998 is presented in Table 1. A statistical description and tabular summary are presented in Appendix A.

Table 1
Wave Statistic Summary

Season	H_{avg}, m	H_{max}, m	T_{mode}, sec	θ_{mode}
Spring	0.68	4.1	8.5	SE
Summer	0.62	2.6	8.5	SE
Fall	0.72	4.2	9.5	ESE
Winter	0.76	3.9	9.5	ESE
Total	0.69	4.2	8.5	ESE

The wave climate offshore of Monmouth Beach is influenced by sheltering from the north by the landmass of Long Island, New York. The short fetch between Long Island and Monmouth Beach limits locally generated wave growth. Directional statistics compiled for the wave record at Long Branch indicate that the most common measured wave direction is from the east-southeast (approximately 35 percent of the total waves measured). Directional characteristics (both seasonal and cumulative) of the measured wave record at Long Branch are presented in Figures 3 through 7. These figures reveal apparent seasonal variations in the wave climate. Waves during spring and summer tend to approach most frequently from the southeast (37 and 43 percent, respectively), whereas wave approaches during the fall and winter tend to be from the east-southeast (40 and 39 percent, respectively).

The shoreline orientation at Monmouth Beach is approximately 4 deg east of north, resulting in a net wave-generated longshore sediment transport to the north, as observed by sand impoundment on the north side of groins in the area. Only 2.3 percent of the total measured wave events show an approach from north of the sector centered about due east. The long-term predominant direction of longshore transport to the north is well known (Caldwell 1966).

Both tropical storms (hurricanes) and extratropical storms (northeasters) are experienced along the north New Jersey coast. The most energetic waves occur during the fall and winter months (when frequent northeasters occur). Spring and summer are typically less energetic, but Atlantic Ocean hurricanes during the months of July through November can contribute large wave heights to the record. The length of the wave record collected at the Long Branch wave gauge is insufficient to characterize the storm climate, but some general observations can be discussed. The largest significant wave height measured at the Long Branch gauge was 4.2 m. The maximum wave height during a given month during the data-collection period ranges from 4.2 m in November to 1.8 m in May. Details of average and maximum wave heights in the 3-year record are given in Appendix A.

Beach Profile

Beach profiles within the hot spot were surveyed as part of beach-fill construction monitoring. Beach profiles are presented at three locations within the hot spot. Figure 2 identifies selected locations of the beach profiles measured within the hot spot area. Beach profiles surveyed during October 1995 (just prior to the emergency fill) within the hot spot are presented in Figures 8 through 10 (note that the photograph of Figure 2 was taken September 1996.) The beach profiles within the hot spot are of similar shape, but become increasingly steep near Groin 44. The steep slope of the beach profile is likely due to the combined influence of the groin and the bathymetric contours to the north, which lie considerably deeper because of the offset in beach planform.

Berm width (elevation 3.5 m NGVD) increases with distance towards Groin 44, from 18 m at Profile 275 to 46 m at Profile 255. Relationships

between vertical datums at Monmouth Beach are presented in Figure 11. Datums based on measured water levels, for example, mlw, are derived from a 6-year tide record measured at the pier in Long Branch, New Jersey.1

Sediment samples were collected within the hot spot between October 1994 and January 1995 at the backshore beach, midberm, mean high water and mean low water. Grain-size analysis (Smith, Gravens, and Smith, in preparation) of the beach-fill material indicates that the median grain size d_{50} is 0.64 mm. Defined by this median grain size, an equilibrium beach profile (Dean 1977, 1991) is determined and plotted for each of the profiles in Figures 8 through 10. Profiles 265 and 275 are approximated well by the equilibrium beach profile; however, Profile 255 is considerably steeper than the equilibrium profile.

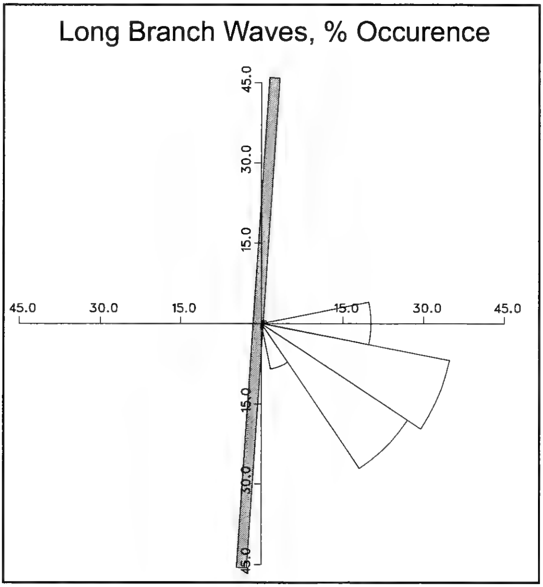


Figure 3. Long Branch, New Jersey, wave rose

[1] National Ocean Service (NOS) web page *http://www.co-ops.nos.noaa.gov.bench/nj/ 8531991.txt*

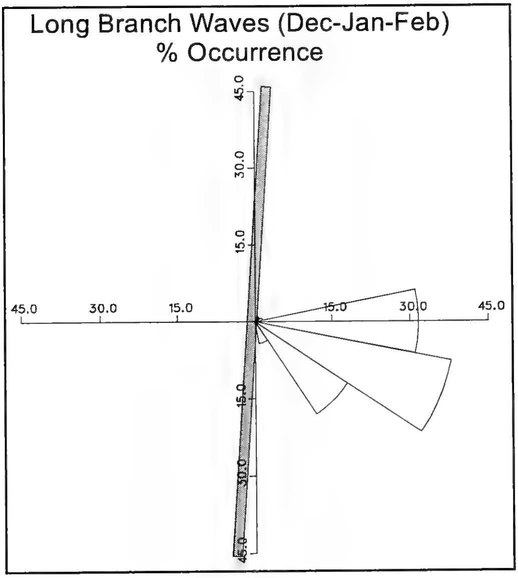


Figure 4. Long Branch, New Jersey, wave rose (winter)

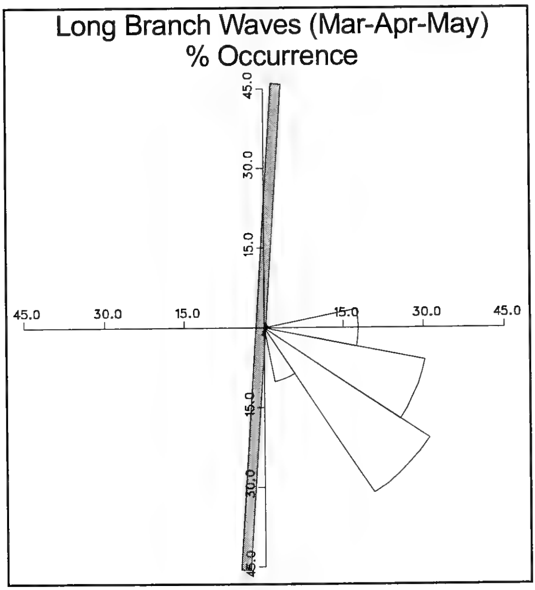


Figure 5. Long Branch, New Jersey, wave rose (spring)

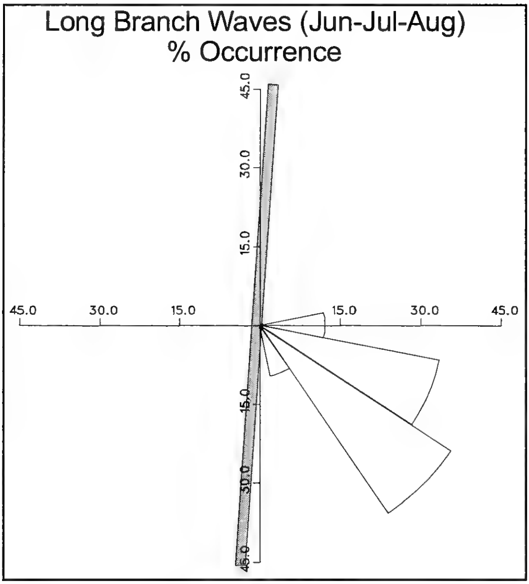


Figure 6. Long Branch, New Jersey, wave rose (summer)

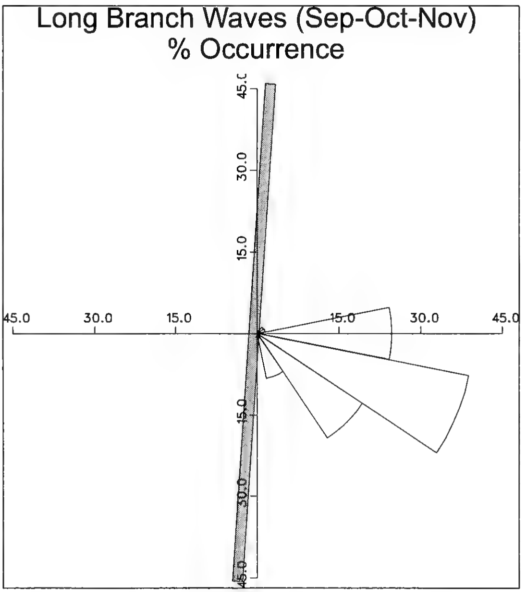


Figure 7. Long Branch, New Jersey, wave rose (fall)

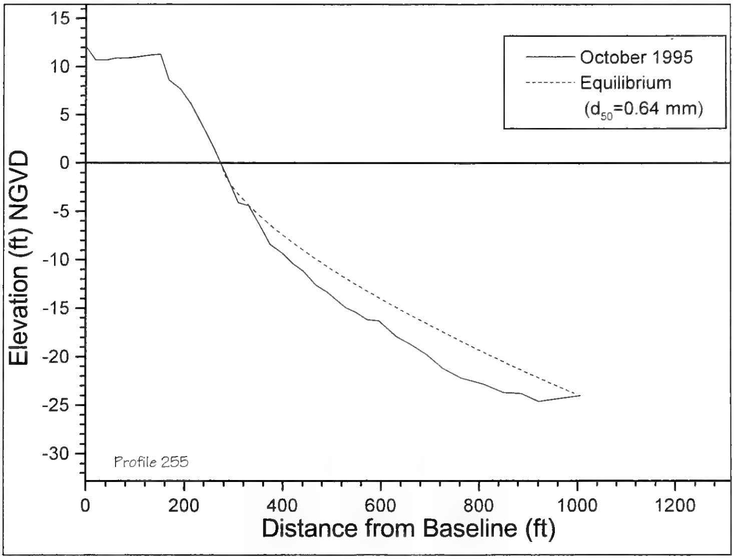


Figure 8. October 1995 Profile 255 with equilibrium profile (d_{50} = 0.64 mm)

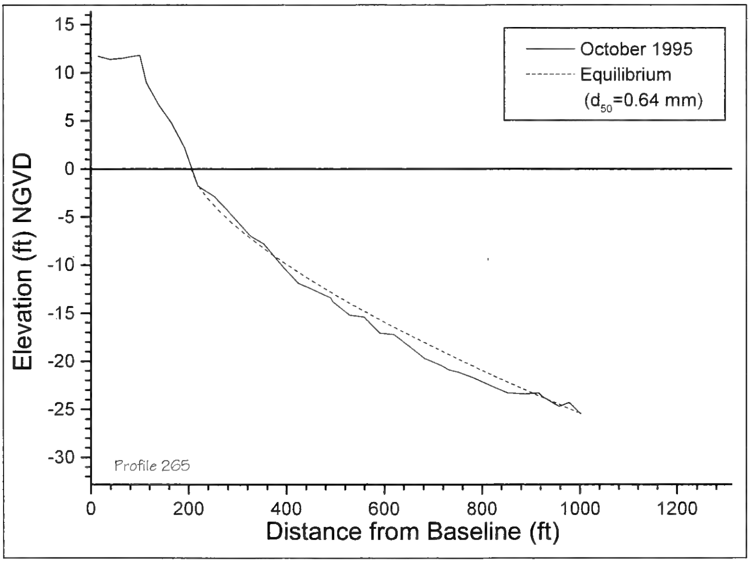


Figure 9. October 1995 Profile 265 with equilibrium profile (d_{50} = 0.64 mm)

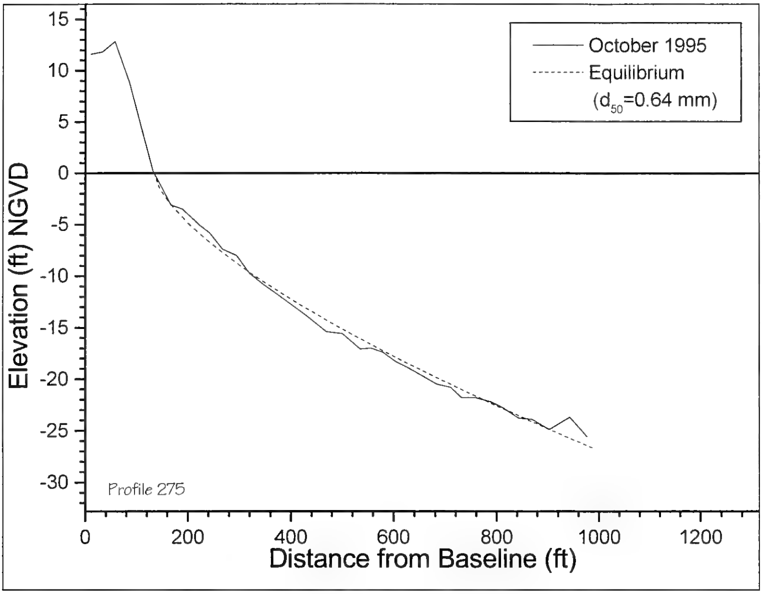


Figure 10. October 1995 Profile 275 with equilibrium profile (d_{50} = 0.64 mm)

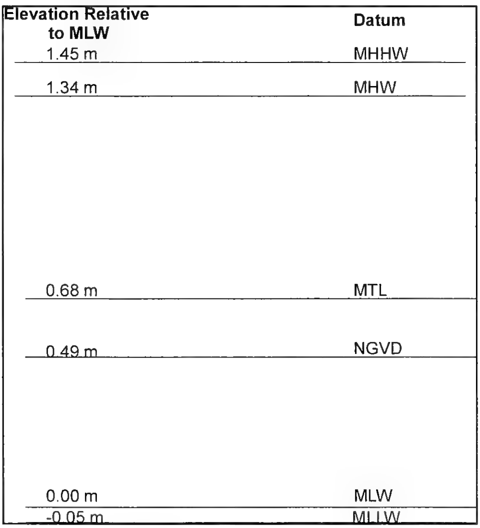


Figure 11. Relative position of vertical datums at Long Branch, New Jersey

3 Functional Design

This chapter describes study procedures and results for a shore-protection functional design aimed to reduce the rate of loss of beach fill at Monmouth Beach. An analysis was performed on the geometrical configuration of the structures at the project site to obtain an empirical criterion relating groin length and spacing between groins. Any design-involving placement of a groin field should be consistent with successful groin performance at the site. Then, the GENESIS numerical model (Hanson and Kraus 1989) was evaluated for describing shoreline change of a beach protected by multiple groins and T-head groins. This evaluation was accomplished by comparing calculated shoreline positions with those measured in a movable-bed physical model.

Two means of specifying boundary conditions at the project for GENESIS were considered, and a regional sediment-budget approach was taken after evaluation of results. Design alternatives were developed based on experience at the site and objective of the study. The alternatives were then examined with the calibrated GENESIS model and refinements made to arrive at two recommended designs for consideration by the New York District. The evaluation was based on a criterion of minimum landward encroachments by the shoreline on the 75-ft (23-m) protective berm width over a 6-year simulation interval. Time series of measurements of the locally incident waves were supplied as forcing conditions for the model.

Empirical Evaluation of Groin Performance

For developing alternatives for the functional design of a groin field at the Monmouth Beach hot spot, aerial photographs taken south of Monmouth Beach were examined to evaluate performance of the existing groin field. The primary objective of the proposed groin field at Monmouth Beach is to maintain a 75-ft storm berm width along the 2,600-ft-long (800-m) hot spot.

The reach of shoreline evaluated ranges between Groins 51 and 104 (Long Branch to Asbury Park city limits) as referenced in the GDM. This reach of shoreline was selected for its close proximity to the hot spot and isolation of the structures within this region from the influence of the beach-fill project constructed to the north. The set of aerial photographs taken 11 October 1997 was analyzed in the evaluation because it represents the most recent aerial photography available for this study. The October 1997 shoreline between

Groins 51-60 is presented in Figure 12. Inspection of aerial photography taken at earlier dates did not reveal significant changes in shoreline position compared with the October 1997 photographs.

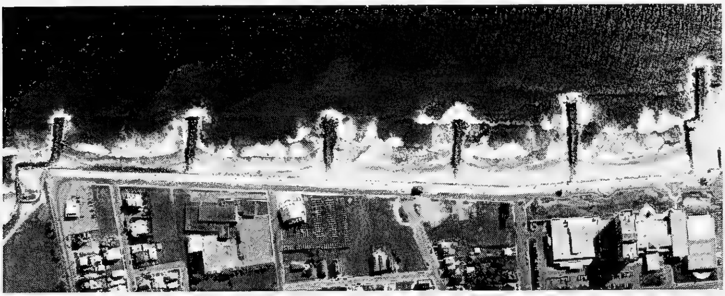


Figure 12. Groin field between Structures 51-60 (11 October 1997)

To quantify groin performance within the evaluated region, groin length, longshore groin spacing, and shoreline position within groin cells were measured. In a review of groin functioning, Kraus, Hanson, and Blomgren (1994) found that the ratio of groin length to the width between them was recommended in the literature to lie between 2 and 4. Kraus et al. also found through numerical modeling with GENESIS that groin permeability and bypassing play central roles in groin sand-retention capability. Bypassing, in turn, depends on the representative wave height and depth at the end of a groin, which is related to groin length and the beach-profile shape.

For the empirical analysis, a baseline typically colocated with the seawall was established between groins. This baseline served as a point of reference for measuring groin length (distance from baseline to waterline at tip of structure) and shoreline position within the groin cells. Shoreline position was measured at five locations within each groin cell, at each structure, and at three equidistantly spaced locations between structures. Two empirical measures, fill ratio and minimum (landward) shoreline position, were selected as criteria to evaluate groin performance.

Estimates of the relative quantity of sand impounded within a groin compartment provide a measure of the effectiveness of adjacent groins. The fill ratio is defined as the ratio of dry beach area within a groin compartment to the area that would result if the groin compartment were completely filled with a dry, straight beach between groin tips, as illustrated in Figure 13. The fill ratio is given by

$$F = \frac{Area_{dry}}{Area_{total}} = \frac{\left(\frac{1}{2}y_1 + y_2 + y_3 + y_4 + \frac{1}{2}y_5\right)}{2(l_1 + l_2)} \qquad (1)$$

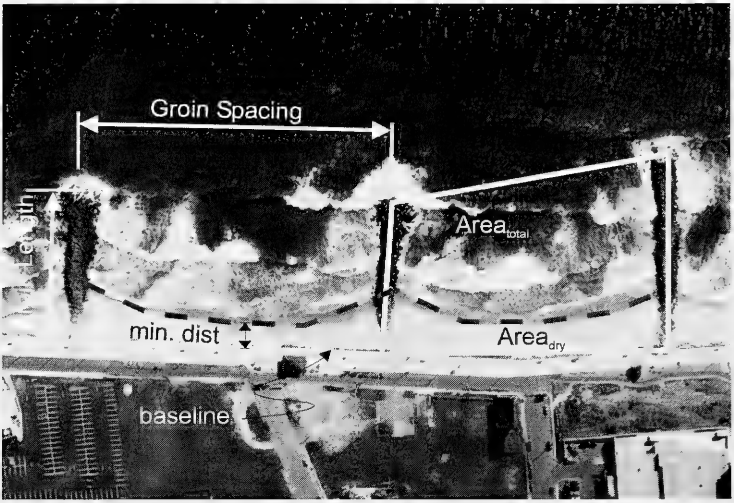


Figure 13. Schematic of performance measures used in empirical analysis

where y represents distance from the established baseline (subscripts 1-5 indicate positions within the groin compartment for trapezoidal rule area calculation), and l represents the length of groins for the respective compartments.

Figure 14 is a plot of the groin spacing/length ratio of compartments 51 to 103 and the associated fill ratios. There is significant scatter in the data, attributed to varying sediment supply (for example, because of local fill operations) and gradients in the longshore sand transport rate. However, a distinct decreasing trend in maximum fill ratio with increasing width-to-length ratio is evident. Width-to-length ratios between 1 and 2 result in maximum fill ratios of approximately 40 percent, whereas width-to-length ratios greater than 3 result in fill ratios on the order of 10-15 percent. Therefore, width-to-length ratios between 1 and 2 are recommended for this section of coast.

Knowledge of the relative amount of fill that a particular groin compartment may retain is helpful information, but not the focus of functional design. The design requirement for this study is the minimum dry beach width of the groin compartments, as defined in Figure 13. Figure 15 presents minimum distance to the baseline (seawall) versus width-length ratio for groin compartments 51-81. Groin compartments south of Groin 83 were excluded from this analysis because of the inconsistent nature of the groin field. The groin field south of Groin 83 is characterized by mixed T-head and linear groins, landward set back of the seawall, and portions of shoreline without seawalls. From Figure 15, width-to-length ratios of 1.5 provide a 75-ft dry beach, assuming a constructed groin field will behave similarly to the existing groins. Groin compartments with minimum berm widths greater than 75 ft are bounded by 300- to 400-ft structures. One alternative presented by CP&E (1997) includes a groin field with 280-ft-long

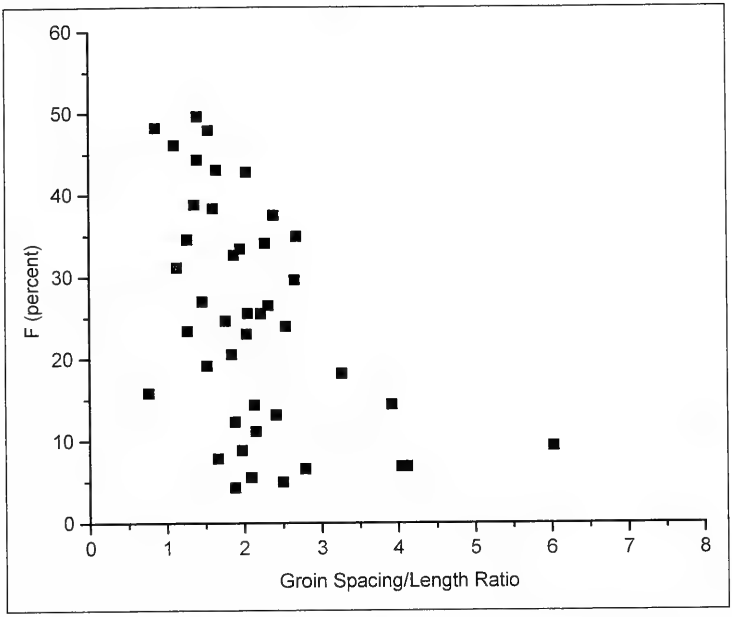


Figure 14. Fill ratio versus groin spacing/length ratio

groins spaced at 450 ft. The width-to-length ratio of this alternative (1.6) and the length of the structures (280 ft) are consistent with the empirical evaluation of groin performance.

Relationships derived from this empirical analysis were consulted to direct the design of proposed structures. Proposed groins will be constructed with lengths on the order of 300 to 400 ft and with width-to-length ratios of approximately 1.5. The design parameters identified here are not considered rigid design requirements. Instead, the performance measures of existing groins serve as guidance in conjunction with sound engineering analysis and judgment towards development of a recommended design.

GENESIS Simulation of Physical Model Results

GENESIS has been successfully applied in numerous evaluations of shoreline change associated with beach fills and coastal structures (e.g., Hanson, Gravens, and Kraus 1988). In the present study, GENESIS capabilities for modeling shoreline response to multiple straight and T-head groins were evaluated by replication of movable-bed laboratory experiments. Kraus (1983) modeled the response of the shoreline to a single detached breakwater in a physical model with the predecessor model to GENESIS. Hanson and Kraus

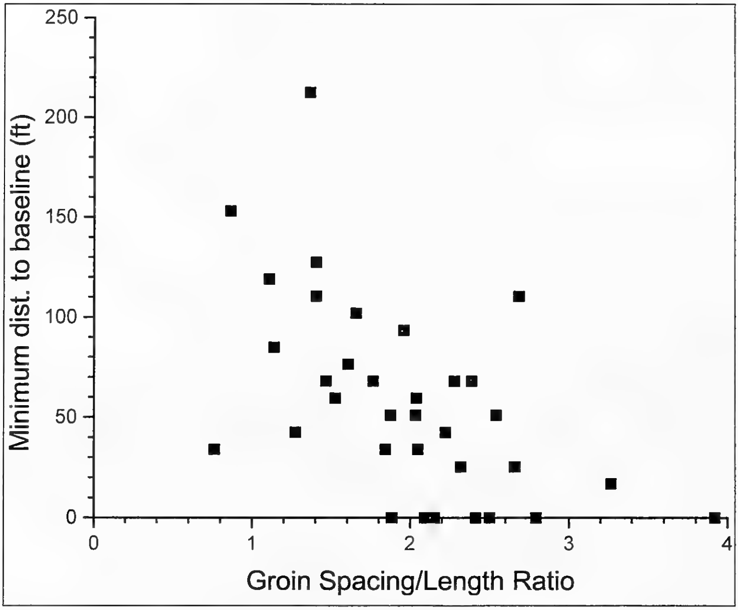


Figure 15. Minimum distance to baseline (seawall) versus groin spacing/length ratio

(1991) applied GENESIS to examine predictions against physical model results for multiple straight groins and shore-parallel detached breakwaters.

Hashimoto, Uda, and Takebuchi (1981) conducted experiments in a large physical model basin (16-m longshore by 20-m cross-shore) to evaluate the functioning of four structural configurations for shore protection from winter-wave conditions on the Pacific coast of Japan. The physical model was constructed at 1:50 scale, and the basin configuration for their Case 4 is shown in Figure 16. The 0.27-mm median grain-size sand was molded into a profile shape representing that of the prototype. The initial beach profile was constructed with a 1/10 slope from the still-water line to a depth of 10 cm, tapering to a 1/70 slope to a distance of 8 m offshore where the slope reverted to 1/10 to a depth of 40 cm.

Cases 1 and 4 of this laboratory study were simulated using GENESIS (Hanson and Kraus (1991) evaluated Cases 1 and 3). Case 1 is configured with three groins extending 1.8 m seaward of the initial shoreline, spaced 6 m apart with the outermost groins located 2 m from the lateral boundaries of the wave basin. The groins were constructed of small concrete blocks that extended well above the water surface and were impermeable to waves. The configuration for Case 4 was similar to that of Case 1 with the addition of 1.2-m-long T-heads to the groins. Incident wave conditions for the physical model tests were

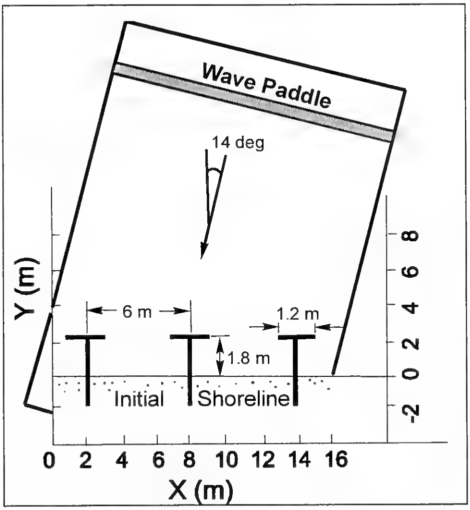


Figure 16. Laboratory arrangement for Case 4 (Hanson and Kraus 1991)

H_o = 5.8 cm, T = 1.2 sec, and θ = -14 deg. The obliquely breaking waves generated a longshore current and longshore sand transport on the model beach.

Calibration

GENESIS was calibrated to Case 1 of the physical model study. Cross-shore profile adjustment is evident in the shoreline positions from the early portion of the physical model simulation. For this reason, the shoreline measured after elapsed time t = 1 hr was used as the initial shoreline for calibration. Calibration resulted in model coefficients of K_1 = 0.3 and K_2 = 0.15 (same values obtained by Hanson and Kraus (1991)). Comparisons of the calibrated shorelines modeled by GENESIS and measured in the physical model basin at t = 1, 6, and 18 hr are presented in Figure 17. The modeled and measured shorelines at elapsed time of 6 hr closely agree. The modeled shorelines at t = 18 hr agree in trends with the measured shorelines, but do not contain the smaller scale features evident in the measured shoreline. These smaller scale features are likely created by basin circulation and reflected wave patterns that are not represented in the numerical model.

Modeling T-head groins

T-head groins are modeled in GENESIS by superimposing a detached breakwater on the tip of a diffracting groin. Using the same incident waves and calibration coefficients from Case 1, the T-head groin configuration was simulated. The resulting shorelines compare favorably with those measured in the physical

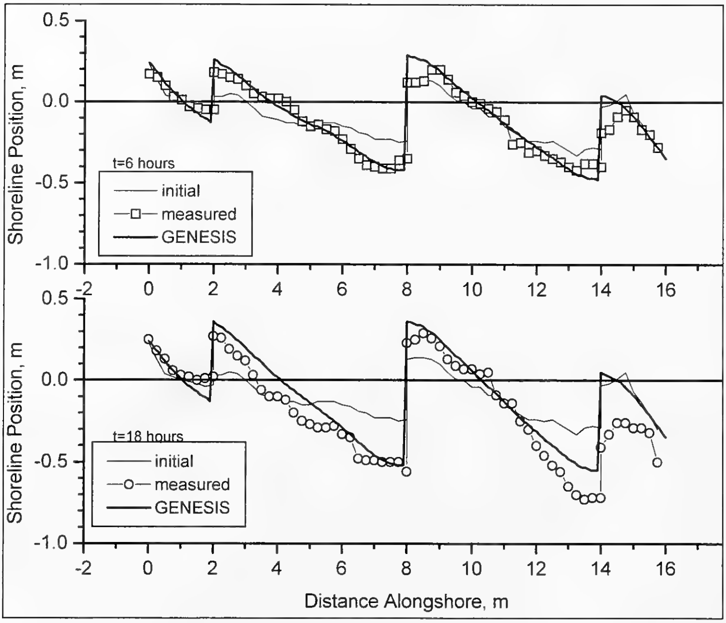


Figure 17. Case 1 physical model/GENESIS comparison

model as shown in Figure 18. However, a noticeable difference exists between the shoreline evolutions of the compartments to the left and right of the center groin (Groin 2). The compartment to the left of Groin 2 is more irregular and cusp-shaped than the compartment to the right of Groin 2. This difference may be explained by the irregular bathymetry that evolved between the groins at longshore coordinates $x = 2$ and 8 m. The bathymetry (from an unpublished report documenting the physical model study) evolved during the experiment to form a transverse bar near the updrift flank of the Groin 1 T-head. Wave energy focused by this transverse bar could be responsible for the greater erosion of the fillet located updrift of Groin 1. The shoreline produced by GENESIS shows better agreement with the shoreline within the compartment to the right of Groin 2. Bathymetric contours within and seaward of this compartment are nearly straight and parallel. The better agreement of GENESIS with the updrift compartment of Groin 2 may be explained by consideration of the action of Snell's Law implemented in the GENESIS internal wave model (which is based on locally straight and parallel contours).

GENESIS demonstrates capability in representing shoreline change influenced by T-head groins with constant wave conditions. In design applications, however, wave conditions are not constant. To assess the behavior of the model under varying wave conditions, the wave direction was sinusoidally varied. With increased wave obliqueness included with these simulations, the shoreline

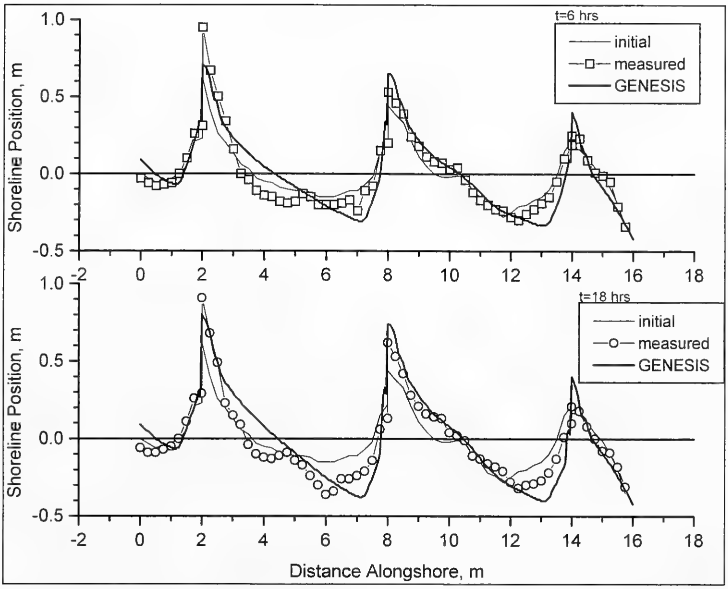


Figure 18. Case 4 physical model/GENESIS comparison

advanced to the tip of the T-head structures, causing the model to cease operation. GENESIS does not allow tombolo formation at detached breakwaters (T-heads in the present case).

GENESIS showed promise in the modeling of shoreline response to T-head groins in comparison to the physical model study. Limitations related to tombolo formation in the present version of GENESIS prevent application of the model to the evaluation of T-head structures in the field. Application and influence of T-head groins will be considered qualitatively in the evaluation of design alternatives.

Model Evaluation of Design Alternatives

With concern over the anomalously high erosion rates of the beach fill at Monmouth Beach, a study evaluating alternatives to mitigate the rapid erosion was undertaken. The purpose of the design was to maintain a protective berm 75-ft wide fronting the seawall for the duration of the 6-year renourishment interval. Effectiveness of each shore-protection design alternative was evaluated with the shoreline-change numerical model GENESIS. Design alternatives were composed of three CP&E (1997) design alternatives and two variations on these designs. Evaluated design alternatives are presented in Table 2.

Table 2 Design Alternatives	
Alternative	Description
1 (CP&E)	No added structures. Allow beach fill to south to nourish hot spot.
2 (CP&E)	Extend Groin 44 350 ft.
3	Extend Groin 44 100 ft.
4 (CP&E)	Add four 280-ft groins between Groins 44 and 45.
5	Add four 100-ft groins between Groins 44 and 45.

The modeled domain was established between Groins 39 and 51 (2.67 miles) with the hot spot nominally centered within this domain. Directional wave measurements collected at 32.8-ft (10-m) depth offshore of the Long Branch pier (near Groin 60) provided wave forcing for the simulations of longshore sand transport and shoreline response to various alternatives. Directional wave measurements were available from July 1995 through March 1998. Gaps in the wave record longer than several days were filled with data from the same date of another year in the record. Wave data from October 1995 to March 1996 were substituted for missing data that occurred during October 1996-March 1997. Also, to extend the record length to 3 complete years, wave data from April 1997 to June 1997 were appended to the March 1998 wave record. GENESIS stepped through this 3-year wave record twice to simulate the 6-year renourishment interval. The general shoreline orientation of the modeled reach is 3.8 deg east of north.

Calibration

Calibration of the shoreline-change model was considered through two methods: (a) calibration to shoreline position and (b) calibration to an accepted sediment budget. Calibration of GENESIS to observed shoreline changes requires historical shorelines and waves representing the nearshore wave conditions during the time of shoreline change. Because GENESIS does not account for cross-shore changes in the beach profile (and consequently no change in shoreline position related to cross-shore transport), the calibration period should be seasonally consistent, having similar starting and ending dates. Simulation of a long record, typically greater than 10 years, reduces uncertainty associated with interannual variability between years even for the same season. The influence of cross-shore beach profile adjustments, such as the initial cross-shore adjustment of a beach fill, should be avoided if calibrating to shoreline position.

An alternative method of calibration is to reproduce an accepted sediment budget. Caldwell (1966) estimated from data on long-term shoreline change for the north New Jersey coast that the average annual net longshore sand transport rate at Asbury Park is 319,000 cu yd/year (244,000 cu m/year) and 493,000 cu yd/year (376,000 cu m/year) to the north along Sandy Hook. Gravens, Scheffner, and Hubertz (1989) estimate potential net transport based on hindcast wave information (including wave sheltering) at Asbury Park to be 249,800 cu yd/year (191,000 cu m/year) and 401,500 cu yd/year (307,000 cu m/year) to the north at

Sandy Hook. Evident in the regional perspective of longshore sand transport is a tendency of increasing northbound transport with distance to the north because of increasing wave sheltering by Long Island. These estimates of the average net longshore sediment transport and longshore transport gradient can serve as a means of calibrating the longshore sand transport model in GENESIS.

Calibration to shoreline. Initial calibration of GENESIS began with comparisons of shoreline change between October 1996 and April 1997. (During this period, the Long Branch directional wave gauge was inoperative, but this period is the only one available in which no shoreline disturbances related to placement of beach fill are evident.) Gated boundary conditions were established at each lateral boundary to represent the prefill conditions south of the hot-spot region. Calibration to shoreline change resulted in a $K_1 = 0.8$ and $K_2 = 0.4$. The resulting annual net longshore sand transport rates at the lateral boundaries are 1,310,000 cu yd/year (1,000,000 cu m/year) to the north at the northern boundary and 209,000 cu yd/year (160,000 cu m/year) to the north at the southern boundary, which are considered excessively high (compare Caldwell 1966).

The transport rate at the north boundary and the transport gradient across the modeled domain are not rational in light of previous estimates of the average-annual transport rate. The overestimation of the calibrated longshore transport rate is probably caused by contamination through cross-shore adjustment of the beach fill by interseasonal differences in the short record. The beach profiles of the initial calibration shoreline (October 1996) were likely in a near-summer condition and steeper than equilibrium because of the December 1995 emergency beach fill. This beach profile would be expected to adjust offshore during the erosive wave conditions of winter, resulting in shoreline recession that can only be accounted for by longshore transport rate in the shoreline change model. Compensating for the cross-shore profile adjustment with the shoreline change model results in the relatively large value of K_1 and, consequently, large longshore transport rates. Because of the irrational results obtained with the calibration to available shoreline positions, calibration to an accepted sediment budget was initiated.

Calibration to sediment budget. In addition to the complications associated with the available shoreline data, lateral boundary conditions representing the prefill condition south of the hot spot are inappropriate for evaluating the functioning of the beach fill after completion of Section I (Figure 1). The gated boundary condition specified in the initial calibration functions to allow a relatively constant flux of sand into the model domain. As fill material placed to the south of the hot spot erodes under exposure to the regional gradient in the transport rate, the supply of sand available for transport north into the model domain is expected to decrease. With this in mind, a moving boundary condition was applied to the south boundary to represent the eroding condition of the beach fill. The initial shoreline position was established to coincide with the shoreline position of the construction template adjusted to the equilibrium beach-profile shape. The rate of shoreline movement at the south boundary was specified such that the shoreline would be eroded to the prefill condition at the end of the beach-fill design life (6 years). Shoreline movement to the prefill condition was selected because movement to the prefill condition is a conservative approach of

modeling the reduction of sand transport with time into the modeling domain. (The longshore transport rate at the boundary is reduced as the shoreline angle becomes less oblique to the incident waves.)

A similar boundary condition was applied to the north boundary, but the difference between the prefill and postfill condition is much smaller at that location. The transport coefficient, K_1, was then adjusted to achieve sand transport rates at the boundaries on the order of values computed by Caldwell (1966) and Gravens, Scheffner, and Hubertz (1989). Final calibration was achieved using $K_1 = 0.55$ and $K_2 = 0.25$. Longshore sand transport rates ranged from 130,000 to 260,000 cu yd/year (99,000 to 199,000 cu m/year) to the north at the south boundary and 525,000 to 650,000 cu yd/year (401,000 to 497,000 cu m/year) to the north at the north boundary. These transport rates are comparable in magnitude, but slightly larger (and therefore conservative) than the values of Caldwell (1966) and Gravens, Scheffner, and Hubertz (1989). Net transport rates for the sediment-rich condition of the nourished beach are expected to be somewhat larger than the rates for the sediment-starved prefill shoreline.

Initial shoreline position. The construction template shoreline is inappropriate to use as an initial shoreline for the GENESIS shoreline evolution simulations. The construction template is typically placed with considerably steeper slopes than the equilibrium profile, and the resultant cross-shore adjustment of the profile is not accounted for by GENESIS. Therefore, an equilibrium profile shape was developed for representing the initial condition (expected shoreline position after adjustment to equilibrium). An analytical method was developed to translate the construction-template shoreline position, accounting for equilibrium profile adjustment. Figure 19 illustrates the concepts of the method developed. First, the construction template is superimposed upon the prefill beach profile. An equilibrium beach profile is constructed through the intersection of the construction template and prefill beach profile. The shoreline advance because of the added volume between equilibrium beach profile 1 (denoted as EBP1 in Figure 19) and the construction template is given by Equation 2.

$$\Delta y = \frac{\Delta V}{(h_* + B)} \tag{2}$$

where

Δy = shoreline advance

ΔV = volume between construction template and EBP1

h_* = depth of closure

B = berm height

Equation 3 is then applied to determine the cross-shore distance between the construction template and the adjusted equilibrium beach profile 2, EPB2,

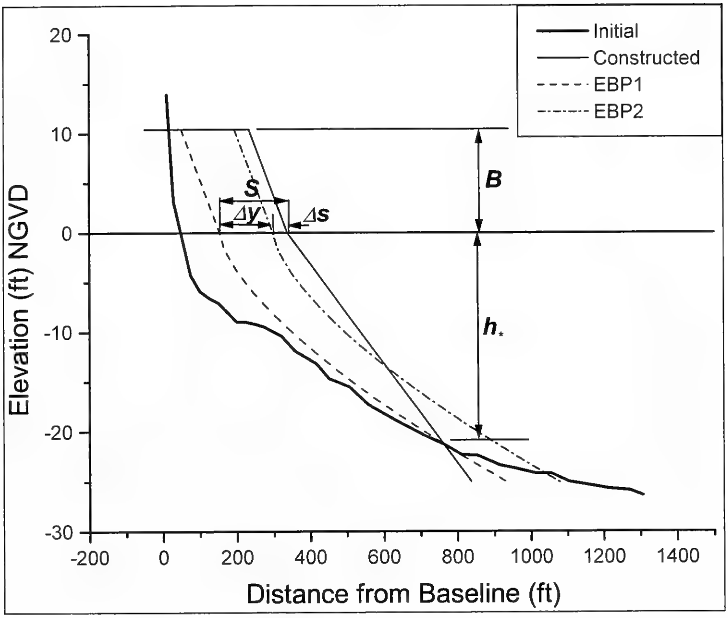


Figure 19. Illustration of method determining initial shoreline position

$$\Delta s = S - \Delta y \tag{3}$$

where

Δs = distance from construction template shoreline to corresponding equilibrium beach profile

S = distance between EBP1 and construction template shoreline

Groin permeability. Structure permeability in GENESIS defines the degree to which longshore sand transport can pass through or over a littoral barrier. The permeability assigned to a structure is difficult to define based on dimensions and characteristics of the structure and is typically determined during model calibration. Permeability of the structures within the modeling domain of this study was determined during calibration by comparison of modeled groin offsets to those recorded in aerial photography. Typical groin permeability resulting from calibration ranged from 0.30 to 0.45. Groin 44 was modeled with zero permeability because its hooked shape and resulting updrift fillet allow little wave action to mobilize sand through the groin.

Design alternatives

Maintaining the boundary conditions and transport coefficients from the final calibration, design alternatives were configured and evaluated with GENESIS. The evaluated design alternatives are summarized in Table 2. The shoreline response to each alternative was simulated with a 6-year wave record developed from the 3-year record of directional waves measured at Long Branch, New Jersey, as described in a previous section.

Alternative 1. This alternative introduces no structural modifications within the hot spot at Monmouth Beach. The CP&E (1997) report specifies nourishment intervals of either 2 or 3 years for this alternative. In keeping with a design objective of the New York District to limit beach fills to the prescribed 6-year nourishment interval, no additional beach fills were included in this alternative. The shoreline response of Alternative 1 is expected to resemble the shoreline response of Contract 1A immediately following beach-fill placement. The model simulation includes the feeding potential of the beach fill constructed south of the hot spot, and the simulated shorelines will differ from the observed shoreline changes because of the influence of this sediment supply.

Animation of the time sequence of shoreline response during the year following beach-fill placement reveals similarities between the modeled response and observed response of the beach fill. The GENESIS simulation indicates rapid erosion of the beach fill between Groins 44 and 45 immediately following placement of the fill. In addition, the region north of Groin 44 experiences an initial shoreline advance. These two simulated qualitative behaviors are observed in aerial photography and beach profile surveys following fill placement under Contract 1A.

Minimum, maximum, and final shoreline positions for Alternative 1 are presented in Figure 20. The shoreline within much of the hot spot recedes landward of the design-specified shoreline at some time during the 6-year simulation. However, monitoring of the minimum shoreline position occurring during the 6-year period is not an adequate methodology for describing the performance of a design alternative. The time during which the target condition (a 75-ft protective berm) is violated accounts for duration of the unwanted condition and is considered a more realistic and accurate measure of the effectiveness of an alternative. Therefore, the percentage of the 6-year simulation in which the modeled shoreline position is located landward of the shoreline required to maintain a 75-ft berm is computed and presented in Figure 21.

The cumulative time in which the simulated shoreline within the hot spot is located landward of the target shoreline is approximately 70 percent of the simulation. Downdrift of Groin 44, the shoreline recedes landward of the design berm approximately 40 percent of the time. Spring and winter have the highest percentages of design berm violations within the hot spot. Spring and summer have the highest percentages of design violations downdrift of Groin 44 because of predominant longshore transport directed to the north and the consequential

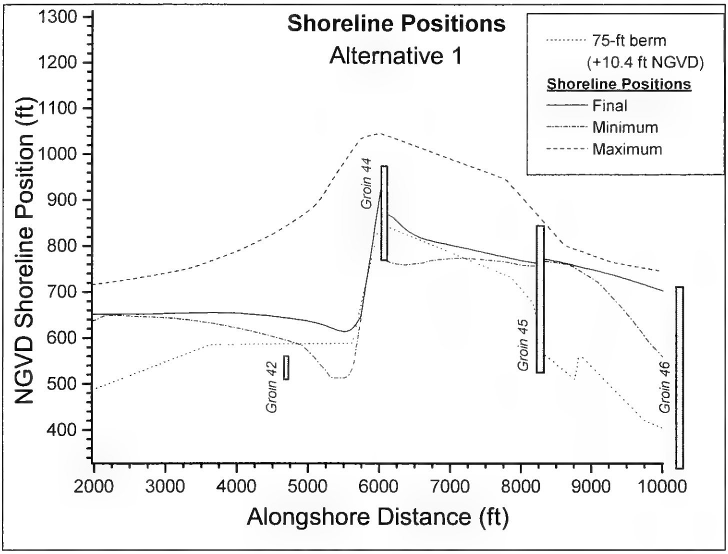


Figure 20. Alternative 1 shoreline positions

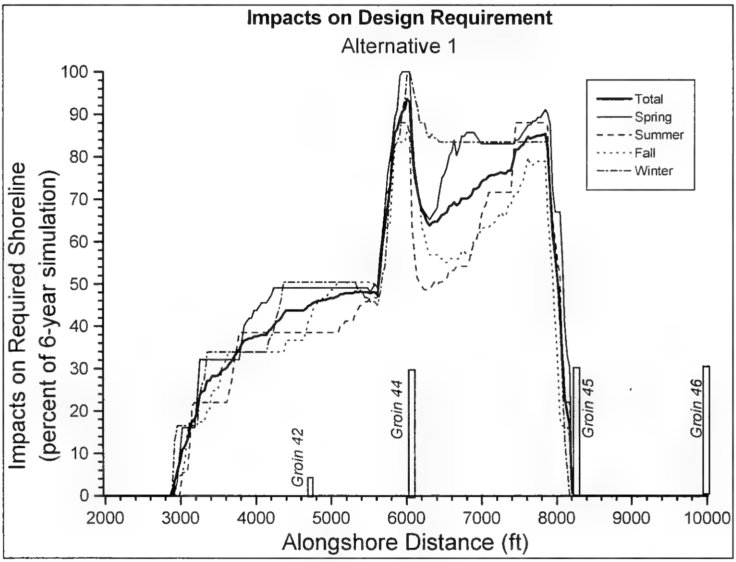


Figure 21. Alternative 1 protective berm impacts

erosion downdrift of Groin 44. The greatest impacts to the protective berm appear within the hot spot, and impacts downdrift of Groin 44 are somewhat smaller.

The sand transported north from the constructed beach fill to the south reduces the rate of erosion experienced at the hot spot prior to completion of the beach fill. However, the 75-ft protective berm within the hot spot experiences erosion during the 6-year renourishment interval. This alternative is attractive in that it requires no additional costs and has the least detrimental influence downdrift of Groin 44. A disadvantage of this alternative is the incomplete protection to the hot spot.

Alternative 2. Alternative 2 is defined by construction of a 350-ft-long extension of Groin 44. The extension is perpendicularly oriented to the seawall and attached to the existing tip of Groin 44. The function of the groin extension is to impound a greater quantity of sand within the area between Groins 44 and 45, providing additional protection to the hot spot.

Structure permeability for extended Groin 44 is expected to vary between the existing and added portions of the structure. The curved nature of the existing structure produces a large fillet and shelters sand adjacent to the groin from the mobilizing forces of waves and runup. The groin extension seaward of the existing tip will be more exposed to the mobilizing forces of waves and runup and therefore is assumed to be more permeable than the existing structure. GENESIS allows only a single value of groin permeability to be assigned to each structure; therefore, an estimate of the composite groin permeability must be assumed for the extended structure. The extended structure should have permeability within the range of existing structures in good condition (0.3) and the nearly impermeable existing Groin 44 (0.0). Simulations of Alternative 2 assume a composite groin permeability of 0.2, but simulations varying permeability between 0.0 and 0.3 were conducted for sensitivity analysis.

The extended groin functions as intended in the model simulations impounding sand a considerable distance seaward of the design berm (Figure 22). However, significantly increased downdrift shoreline recession because of the groin extension is evident. During the 6-year simulation, the shoreline erodes to the seawall for a distance of approximately 2,500 ft downdrift of Groin 44.

Figure 23 indicates that the shoreline remains seaward of the design berm limits within the hot spot for nearly the entire 6-year simulation. The downdrift shoreline, however, is eroded beyond the design berm limits between 70 and 80 percent of the 6-year simulation to a distance 2,000 ft north of Groin 44. Although this alternative meets the requirement of maintaining a protective berm within the Monmouth Beach hot spot, the significant downdrift recession induced by the groin extension may not be considered acceptable.

Alternative 3. Alternative 3 is a variation on Alternative 2 in which Groin 44 is extended 100 ft. This alternative is proposed to retain the protective berm at the hot spot, while reducing the detrimental downdrift impacts found for Alternative 2. Alternative 3 used a composite structure permeability of 0.2 for

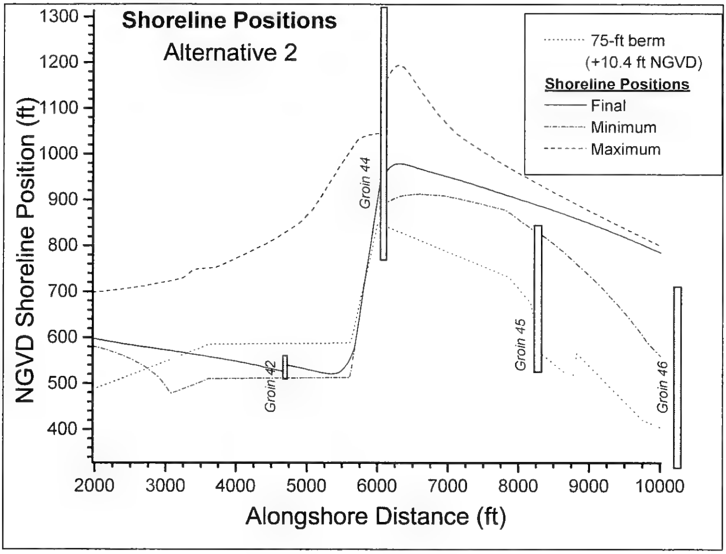


Figure 22. Alternative 2 shoreline positions

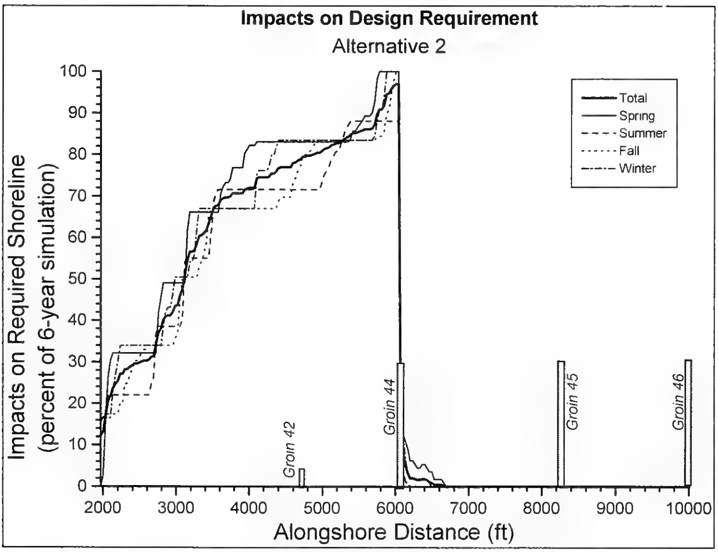


Figure 23. Alternative 2 protective berm impacts

extended Groin 44 (as discussed for Alternative 2.) The reduced downdrift impacts of Alternative 3 are evident in Figure 24, where the minimum shoreline position erodes to the seawall for a distance of 1,150 ft, much less than the 2,500 ft of Alternative 2. Shoreline recession impacting the 75-ft protective berm is presented in Figure 25. Alternative 3 maintains the 75-ft protective berm at the hot spot with the modeled shoreline being located landward of the 75-ft protective berm less than 20 percent of the 6-year simulation. In addition, Alternative 3 has reduced spatial and temporal extents of downdrift erosion compared with Alternative 2. Impacts to the protective berm occur to a distance of 3,600 ft downdrift of Groin 44 with the maximum shoreline recession occurring approximately 70 percent of the 6-year renourishment interval immediately downdrift of Groin 44.

Alternative 3 yields promising results, limiting impacts to the 75-ft protective berm within the hot spot during the 6-year renourishment interval with only small increases in downdrift erosion over Alternative 1. These results are obtained with the relatively minor expense of extending existing Groin 44.

Alternative 4. Alternative 4 proposes the addition of four, 280-ft groins between Groins 44 and 45. The function of these groins is similar to that of Alternatives 2 and 3, to impound additional material at the hot spot. The minimum shoreline position presented in Figure 26 is located landward of the target shoreline within the compartments between Groin 44 and Groin 44c (third groin updrift of Groin 44). The target shoreline encroachment in the first compartment updrift of Groin 44 occurs approximately 65 percent of the 6-year simulation (Figure 27). Observing animations of the calculated shoreline change for Alternative 4, the shoreline erosion within the groin compartments updrift of Groin 44 occurs during reversals in longshore transport direction.

Erosion downdrift of the added groin field results in the minimum shoreline position landward of the target shoreline for a distance of 2,300 ft downdrift of Groin 44 over the 6-year simulation. Target shoreline encroachments occur most often in winter and spring and least often in fall and summer. Downdrift impacts of Alternative 4 are slightly less severe than those of Alternative 2 and remain considerably larger than the impacts of Alternatives 1 and 3. Encroachment of the shoreline near Groin 44 occurs 70-80 percent of the 6-year simulation. The least severe downdrift impacts appear to occur during the winter months, when reversals in longshore transport act to supply sand to the area. This alternative has relatively high cost and causes substantial downdrift recession. Therefore, it is not recommended.

Alternative 5. Alternative 5 decreases the groin length of Alternative 4 from 280 ft to 100 ft with the intent of maintaining the protective berm at the hot spot and reducing downdrift erosion. Figure 28 presents the maximum, minimum, and final shoreline positions for Alternative 5 indicating that the protective berm within the hot spot is eroded within the four groin compartments south of Groin 44. Maximum percentages of protective berm impacts (Figure 29) within the hot spot range from 15 to 75 percent of the 6-year simulation, increasing with distance towards Groin 44. Downdrift impacts for Alternative 5 are reduced to the same levels as Alternative 3, approximately 40-50 percent of the 6-year

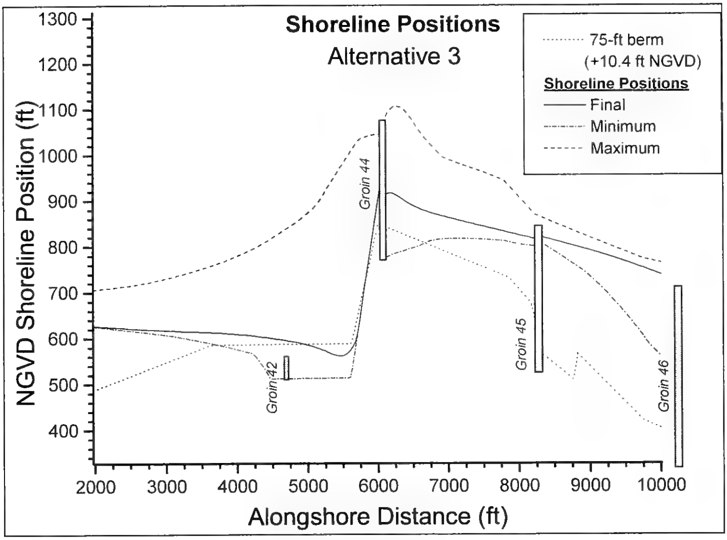


Figure 24. Alternative 3 shoreline positions

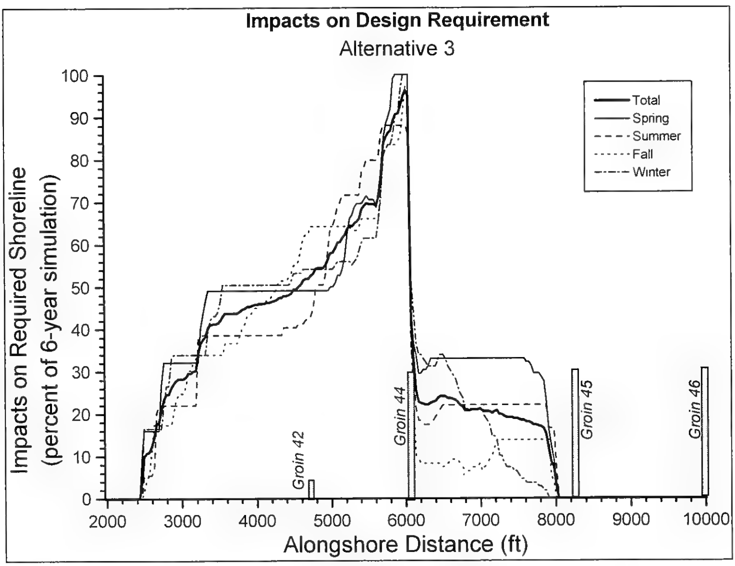


Figure 25. Alternative 3 protective berm impacts

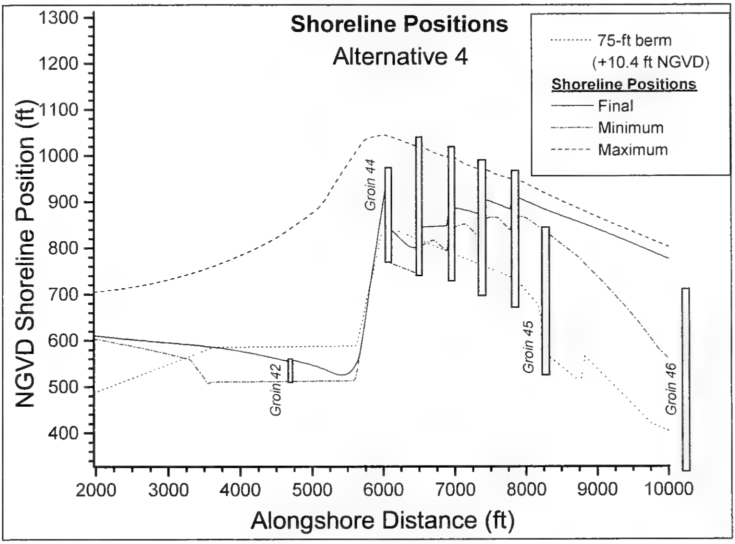


Figure 26. Alternative 4 shoreline positions

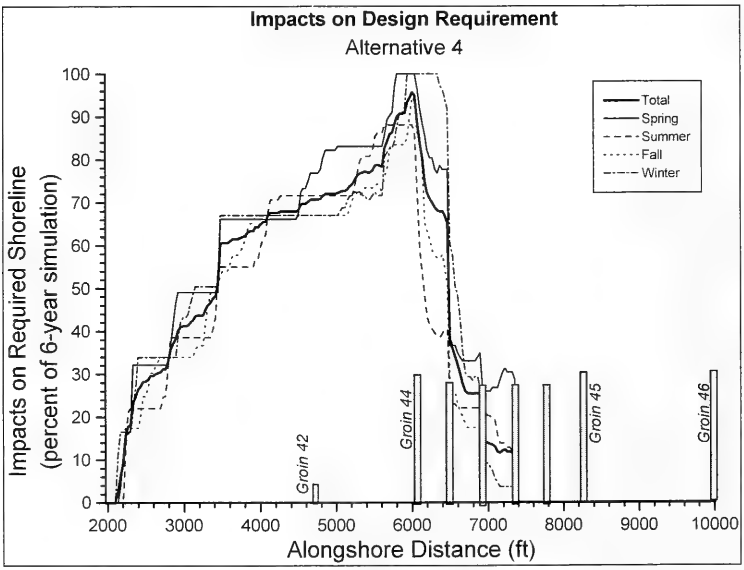


Figure 27. Alternative 4 protective berm impacts

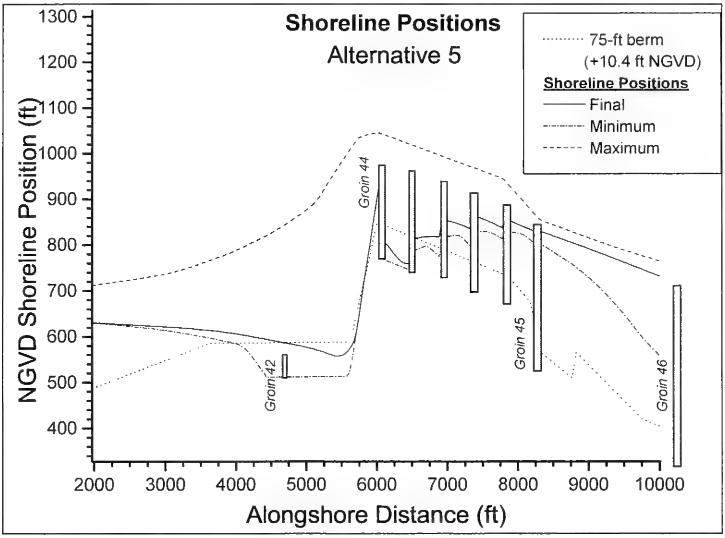


Figure 28. Alternative 5 shoreline positions

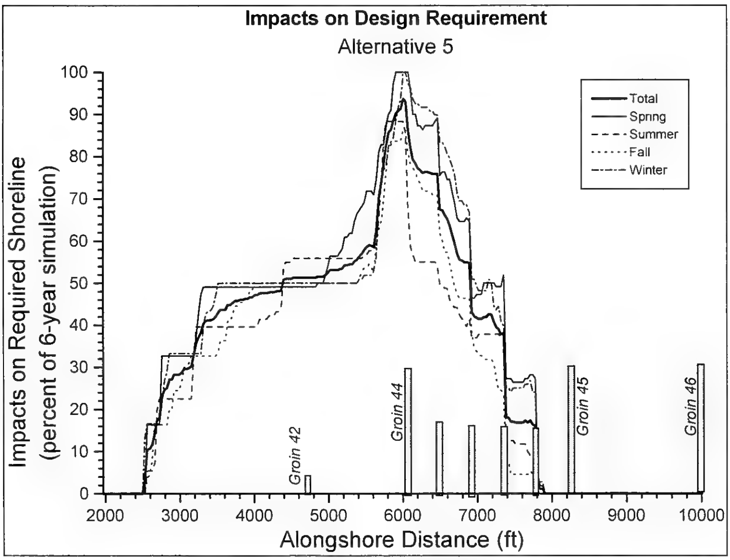


Figure 29. Alternative 5 protective berm impacts

simulation. A simulation with groin lengths increased from 100 to 200 ft resulted in a slight increase in benefits to the hot spot, offset by significant increases in recession of the downdrift shoreline.

Alternative 5 (with 100-ft-long groins) offers less protection to the hot spot than Alternative 3 at significantly greater cost. Therefore, Alternative 5 is not recommended on the basis of the small return on investment compared with other options.

T-head groins. Although an alternative consisting of T-head groins could not be modeled by the present form of GENESIS, qualitative assumptions can be made about the functioning of T-head groins in the configurations of Alternatives 4 and 5. Because T-head structures have a greater capacity to retain sediments within the groin compartment, substituting T-head structures into Alternatives 4 and 5 is expected to reduce the impacts to the protective berm within the hot spot. However, the enhanced trapping capacity of the T-head structures is also expected to increase impacts to the 75-ft berm downdrift of Groin 44.

T-head groins substituted into Alternatives 4 and 5 are expected to offer additional protection to the hot spot as compared with straight groins. However, T-head groins are not a recommended alternative because of increased construction costs and detrimental impacts to the downdrift shoreline.

Recommended alternatives

Alternatives 1 and 3 are considered the preferred alternatives based on the present analysis. Each alternative offers specific advantages over the other, and each tends to meet the design objectives in different ways. Therefore, this study will present both alternatives and define the advantages of each approach. Figures 30 and 31, respectively, show direct comparisons between Alternatives 1 and 3.

Alternative 1 (no additional structures) minimizes erosion of the 75-ft protective berm downdrift of Groin 44, while allowing erosion of the protective berm there approximately 30-50 percent of the 6-year renourishment interval. Erosion of the protective berm within the hot spot occurs approximately 65-80 percent of the 6-year simulation. Alternative 1 has economic and aesthetic advantages because additional structures are not required.

Alternative 3 (100-ft seaward extension of Groin 44) meets the design objective by decreasing berm impacts within the hot spot to less than 20 percent of the 6-year simulation. Erosion of the protective berm downdrift of Groin 44 increases over Alternative 1 and extends approximately 3,200 ft north of Groin 44 because of the increased impoundment of material by the groin extension. The downdrift erosion for this alternative, although greater than Alternative 1, is less than any other alternative involving structural modifications to the existing conditions. Sand tightening of the existing portion of Groin 44 is not necessary because sediments adjacent to the groin are sheltered from mobilizing forces.

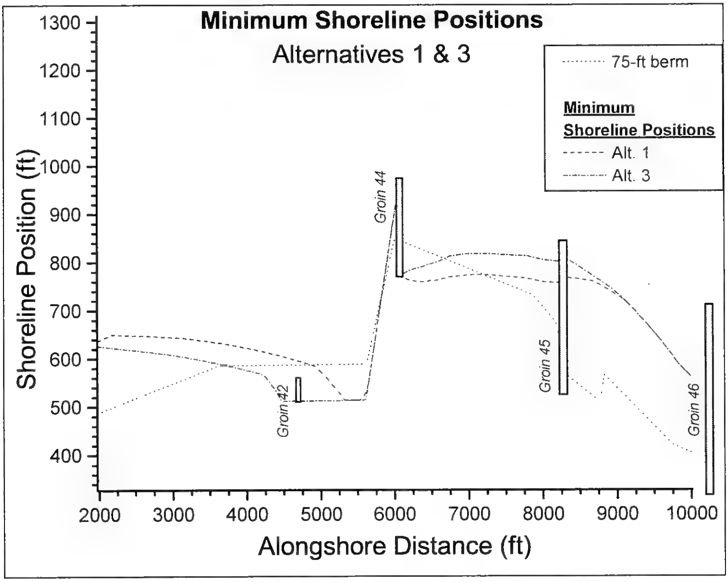


Figure 30. Shoreline comparisons between Alternatives 1 and 3

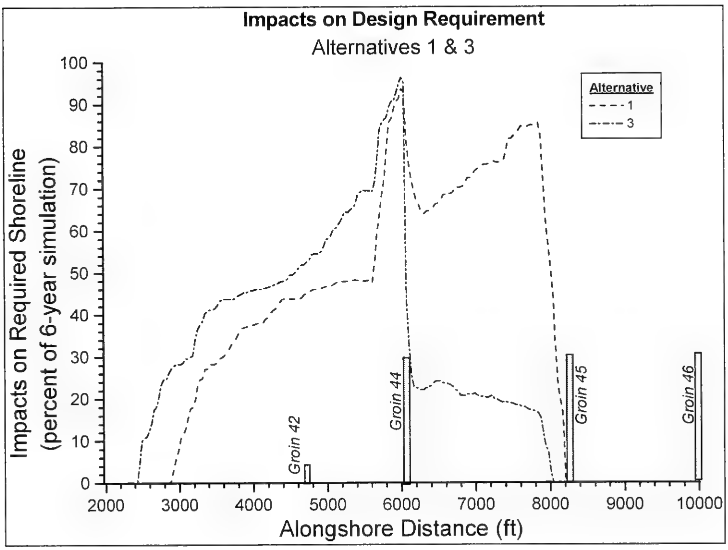


Figure 31. Protective berm impacts for Alternatives 1 and 3

Variability

Deterministic evaluation of design alternatives does not account for the intrinsic variability of the environmental forcing, in this case variability in the wave climate. The 3-year wave record is insufficient to define the long-term wave and longshore transport climate, but considered adequate for characterizing responses of the recommended alternatives to measured variation in wave conditions.

Wave measurements at the Long Branch, New Jersey, DWG show distinct annual and seasonal variability. Annual potential longshore sand transport rates were calculated for the 3-year wave record. Northward-directed transport accounted for 93 percent of the gross transport volume for the 3-year record. From the 1996 wave record, potential transport to the north was slightly greater than average, and potential transport to the south was slightly less than average. Northward-directed transport from the 1996 record accounted for 95 percent of gross transport volume.

Northward-directed transport during the summer 1997 through spring 1998 wave record was of similar magnitude as the 1996 record, but southward-directed transport was greater than average by a factor of 1.6 (indicating more transport reversals). Northward-directed transport accounts for 89 percent of the gross transport volume. Differences evident in the 1996 and 1997/98 wave records are useful in evaluating the influence of wave climate variations on the recommended solutions. Therefore, the 1996 and 1997/98 wave records were repeated through 6-year simulations to estimate variability for Alternatives 1 and 3.

Figures 32 and 33 present the cumulative impacts to the 75-ft berm for the 3-year record and repeated 1996 and 1997/98 wave records for Alternatives 1 and 3, respectively. Note that the dominant longshore transport (to the north) during 1996 increases impacts to the protective berm downdrift of Groin 44. Conversely, transport reversals (to the south) during 1997/98 cause increased impacts to the protective berm within the hot spot. When wave conditions cause significant sediment transport reversals (1997/98), the recommended alternatives differ little in regard to protection of the hot spot. When wave conditions generate persistent transport to the north, erosion downdrift of Groin 44 is less severe for Alternative 1. Alternative 3 provides more protection to the hot spot, whereas Alternative 1 causes less detrimental impacts to the area downdrift of Groin 44.

The variability analysis presents responses of the recommended alternatives to plausible extremes of the 3-year wave record. The 3-year record is insufficient to quantify the full variability of the wave climate at Monmouth Beach. However, the extremes in the transport rate from the available wave data provide guidance on the potential variation and qualitative response of the candidate alternatives to characteristics of the transport rate. The extremes in annual transport rate occurring in the 3-year record were cycled for 6 years to define the variable shoreline responses. It is unlikely that biases in the longshore sand transport rate selected for the variability analysis would reoccur for 6 consecutive years.

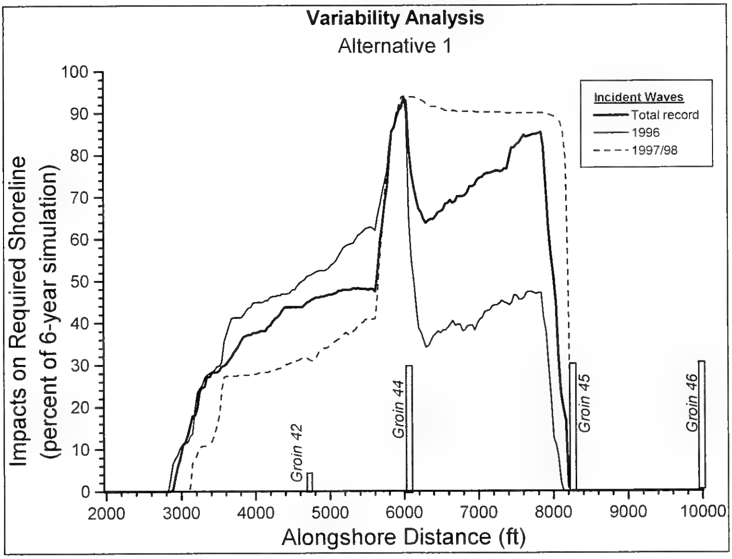


Figure 32. Variability estimate for Alternative 1

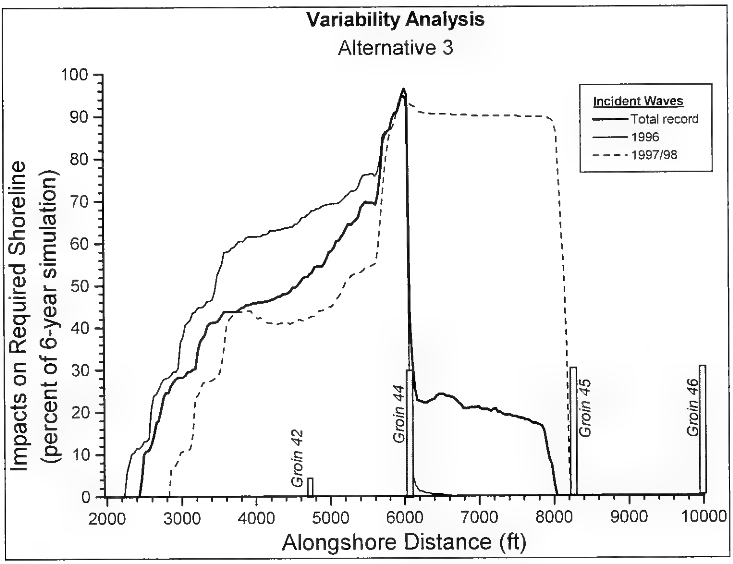


Figure 33. Variability estimate for Alternative 3

Summary and Conclusions

The effectiveness of five design alternatives for mitigating the hot spot at Monmouth Beach, New Jersey, was evaluated with the numerical shoreline change model GENESIS. The objective of the designs was to maintain a 75-ft protective berm fronting the seawall within and adjacent to the hot spot at Monmouth Beach during a 6-year renourishment interval. A 3-year record of waves measured at Long Branch, New Jersey (near the hot spot), served as the environmental forcing for the 6-year simulations. Performance of each design alternative is summarized in Table 3.

Two design alternatives are recommended for consideration by the New York District. Alternative 1 (no new structures) allows beach-fill material placed to the south to nourish the hot spot. Simulations using the 3-year measured wave record suggest that the protective berm width within the hot spot will be less than 75 ft approximately for 60-80 percent of the 6-year renourishment interval. The strength of this alternative is that erosion of the 75-ft protective berm downdrift of Groin 44 is minimized (40-50 percent of the 6-year renourishment interval). In addition, this alternative has economic and aesthetic advantages in that no additional structures must be placed within the hot spot.

Alternative 3 (extending Groin 44 seaward by 100 ft) impounds additional material within the hot spot and reduces impacts to the protective berm. By extending Groin 44 100 ft, erosion of the 75-ft protective berm within the hot spot is reduced to approximately 20 percent of the 6-year renourishment interval. One disadvantage of Alternative 3 is the increased erosion downdrift of Groin 44. For a distance of approximately 3,600 ft north of Groin 44, the 75-ft protective berm is impacted by shoreline erosion. Simulations with the 3-year measured wave record indicate that the protective berm immediately north of Groin 44 will be less than 75 ft in width approximately 40-60 percent of the 6-year renourishment interval. Cumulative berm impacts decrease with distance north of Groin 44.

Table 3
Design Alternatives Considered and Associated Impacts

Alternative	Description	Hot-Spot Impacts, %	Downdrift Impacts, %
1	No added structures. Allow beach fill south of hot spot to supply sand.	60-80	30-45
2	Extend Groin 44 350 ft.	< 5	60-80
3	Extend Groin 44 100 ft.	20	40-60
4	Add four 85-m groins between Groins 44 and 45.	15-70	30-70
5	Add four 30-m groins between Groins 44 and 45.	15-75	30-55

Note: Impacts are defined as a percentage of 6-year simulation in which the 75-ft protective berm would be eroded.

A variability analysis incorporating annual extremes in longshore sand transport as obtained from the 3-year wave record reveals that protective berm impacts at the hot spot are more severe for years in which the percentage of

transport to the south is greater than normal. Conversely, greater erosion downdrift of Groin 44 is anticipated for years in which the percentage of transport to the north is greater than normal. Alternative 3 is the recommended alternative for protecting the hot spot, although Alternative 1 produces marginally fewer detrimental impacts to the area downdrift of Groin 44.

4 Structure Design

The objective of this study is to examine various approaches (including strucures) for extending the beach-renourishment cycle for the Monmouth Beach hot spot to the same 6-year cycle as the adjacent beaches. Of the alternatives exained that accomplished this objective, Alternative 3 involved structural additions and modifications. This chapter reviews the existing structures along the Monmouth Beach shore and describes the armor-unit design and structure cross section as specified in Alternative 3, a 100-ft seaward extension at Groin 44.

Existing Structures

Groin performance (impoundment and bypassing) is primarily a function of the ratio of net and gross longshore sediment transport rates, groin length (parameterized by the ratio of the depth at the toe of the structure and a representative wave height), and structure permeability and crest elevation (Kraus, Hanson, and Blomgren 1994). Length controls bypassing; permeability controls through-passing; and crest elevation controls overtopping of sediment. Structure condition also enters consideration of groin performance because if stones break, deteriorate, or shift, the groin length, permeability, and crest elevation may change and thereby reduce groin effectiveness. Because of the many variables involved in groin functional design, inspection of the performance and structural integrity of groins at or near a project site can provide reliable information for achieving a successful design.

Groin 44 appears to be impounding sand updrift (to the south) as intended (Figures 34 and 35). Groin 46 also impounded sand until the fall of 1997 when it was notched (a 100-ft-long (at mlw line), 7-ft-deep section of the central portion of the groin was removed) to increase its permeability. Since then, no sand has been impounded. Little influence on shoreline position by the notching is evident at the remaining groins (e.g., Groin 45, Figure 36).

The relation between structure condition (stone breakage, deterioration, or shifting) and structure performance at Monmouth Beach is not known. Though Groin 44 is impounding sand, its "fair" condition gives no indication of the existence of any broken or deteriorated stones. Groin 46 was performing as

Figure 34. Groin 44 in March 1998

Figure 35. Aerial photograph of Groin 44 taken in April 1994

Figure 36. Groin 45 in March 1998

originally designed, but after notching, broken armor and core units were discovered in the structure.[1]

A survey of existing structures along Sea Bright to Ocean Township conducted in August 1985 reported the description and condition of structures for the New York District's Sea Bright to Ocean Township General Design Memorandum (New York District 1989). Six of these structures are described in Table 4. Information is not available to determine whether the reported stone sizes represent those specified in the original design of the structure. The stone breakage observed in 1997 at Groin 46 was not noted in New York District (1989). Breakage at other groins may be reflected in the "poor" condition classifying Groins 42 and 43.

The difference in permeability between the existing Groin 44, which is assumed to be nearly zero (see discussion in Chapter 3), and the proposed extension (which will have an estimated composite permeability of 0.20) should have no impact on overall structure function. No major modifications (in particular, sand tightening) to the existing structure should be required prior to construction of the extension.

[1] Personal Communication, 19 March 1998, Ms. Lynn Bocamazo, Civil Engineer, New York District.

Table 4
Monmouth Beach Groins (modified from New York District 1989)

Structure Number	Material[1]	Length ft	Average Crest Elevation ft NGVD	Crest Width ft	Stone Size ft/ton[2]	Condition	Year Built
41	1,2,3	350	10.8	10-15	4-6/10.3	Fair	---
42	1,2	110	11.1	12-14	4-5/7.5	Poor	1923
43	1,2	80	5.4	10	3-4/3.5	Poor	---
44	1,3,4	710	10.9	10-15	4-6/10.3	Fair	1924
45	1,2,3,4	360	6.5	15	3-5/5.3	Fair	1966
46	1,2,4	490	8.0	12	4-6/10.3	Fair	1927

[1] 1-stone, 2-timber, 3-steel, 4-concrete.
[2] $W = D^3\gamma$ where γ = 165 lb/cu ft.

Structure Design

The 100-ft extension of Groin 44 (Alternative 3) will be added to the seaward end of the existing structure, directed perpendicular to the updrift shoreline. The present crest elevation of the seaward end (+11.0 ft NGVD) will be extended for the groin extension for 33 ft, connecting to a sloping section (approximately 1:6.6) for 33 ft, and terminated by the seaward horizontal section at +6.0 ft NGVD. Side slopes will be 1:2.

Design wave

Wave conditions for specification of structure stone size are depth-limited breaking waves determined from the nearshore slope adjacent to Groin 44 and from three storm-surge water levels (10-, 50- and 100-year recurrence intervals) (New York District 1989). The June 1994 (prefill) Profile 255 (Smith, Gravens, and Smith, in preparation) was selected as the design condition because it represents a reasonable eroded condition that maximizes the water depth and breaking wave height at the structure. Figure 37 shows the June 1994 profile, the equilibrium nearshore profile, the center-line cross section of Groin 44, the proposed Groin 44 extension, and the three water-level elevations. From this figure, the depth at the exposed part of the structure relative to each water level is identified for calculating the depth-limited breaking wave. Table 5 lists the water-level elevation, profile elevation, and water depth for each condition.

The maximum, depth-limited wave height for each water level was determined from the water depth of each design water level and wave data from the Long Branch gauge. Ten periods (midvalue in each of the 1-sec period bands ranging from 3 to 13 sec) from the Long Branch wave gauge (see Appendix A) were selected for calculating the depth-limited breaking wave height. The

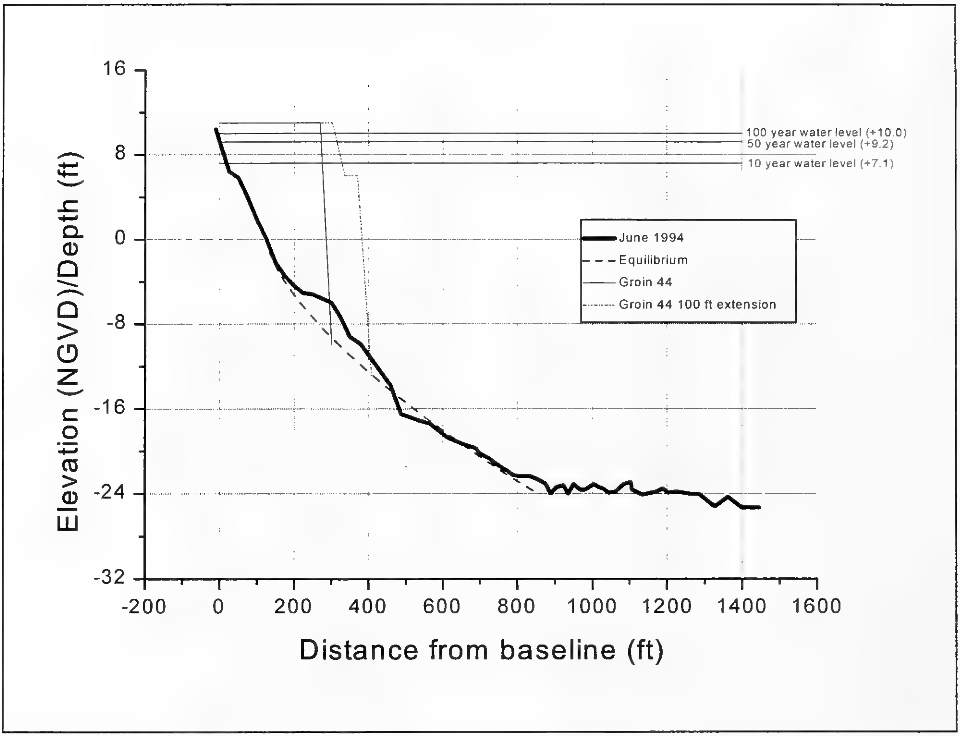


Figure 37. Profile 255 in June 1994 with Groin 44 and proposed extension shown relative to the 10-, 50-, and 100-year water levels

Table 5
Water-Level and Profile Elevations and Structure Depth

Recurrence Interval	Water-Level Elevation ft NGVD	Profile Elevation ft NGVD	Water Depth Ft
10	+7.1	-10.3	17.4
50	+9.2	-9.8	19.0
100	+10.0	-9.6	19.6
Note: For conversion between NGVD and other water-level datums (e.g., mlw), see Figure 11.			

methodology for computing the maximum breaker height follows that presented in the Shore Protection Manual (SPM) (1984) as directed in Engineer Manual 1110-2-2904 (U.S. Army Corps of Engineers (USACE) 1986).

The term d_s/gT^2 is calculated for each wave period for use in Figure 7-4 in the SPM.

where

d_s = depth of water at structure toe

g = acceleration because of gravity

T = wave period

The corresponding H_b/d_s (where H_b is the breaking wave height) is found by interpolating between nearshore slopes 0.03 and 0.04 for Profile 255's nearshore slope of 0.0357 (1:28). Multiplying H_b/d_s by water depth (from Table 5) gives the depth-limited breaking wave height. This approach, rather than the common $d_b = 1.3H_b$ criterion (where d_b is breaking depth), is preferred for structure design because it accounts for nearshore slope and for the distance traveled between initiation of breaking and wave impact.

USACE (1986) specifies use of H_{10} (average of the highest 10 percent of all waves) for design of rubble-mound structures. Depth-limited breaking wave heights calculated as described above are representative of H_{max} and must be converted to H_{10} for calculation of stone size. For conversion, it is assumed that the H_{max} calculated above is approximately equal to H_2 (average of the highest 2 percent of all waves). On the assumption that wave height is described by a Rayleigh distribution, this H_2 was converted to an H_{rms} and then to an H_{10}. Table 6 lists the depth-limited breaking wave height (H_b), H_{rms}, and H_{10} by period and water level. Note that the largest H_b occurs for 12.5-sec periods for all water levels.

Table 6
Design Wave Heights

Period Sec	10-Year Water Level			50-Year Water Level			100-Year Water Level		
	H_b ft	H_{rms} ft	H_{10} ft	H_b ft	H_{rms} ft	H_{10} ft	H_b ft	H_{rms} ft	H_{10} ft
3.5	13.1	5.93	10.6	14.3	6.48	11.5	14.7	6.68	11.9
4.5	13.1	5.93	10.6	14.3	6.48	11.5	14.7	6.68	11.9
5.5	13.9	6.33	11.3	14.3	6.48	11.5	14.7	6.68	11.9
6.5	15.7	7.12	12.7	16.5	7.51	13.4	17.1	7.75	13.8
7.5	16.4	7.43	13.2	17.9	8.12	14.5	17.8	8.11	14.4
8.5	17.9	8.15	14.5	18.6	8.46	15.1	19.2	8.73	15.5
9.5	18.3	8.30	14.8	19.6	8.90	15.8	20.2	9.18	16.3
10.5	18.6	8.46	15.1	20.3	9.24	16.4	20.6	9.35	16.7
11.5	19.1	8.70	15.5	20.9	9.50	16.9	21.0	9.53	17.0
12.5	19.3	8.78	15.6	20.9	9.50	16.9	21.6	9.80	17.4

Calculation of stone size and damage potential

The USACE standard for sizing armor units for breakwaters, jetties, and groins is the Hudson formula (SPM 1984; USACE 1986). The Hudson formula (Equation 4) was developed in the 1950s based on the results from small- and large-scale physical models conducted at WES (SPM 1984).

$$W = \frac{\gamma_r H^3}{K_D \left(\frac{\gamma_r}{\gamma} - 1\right)^3 \cot\theta} \tag{4}$$

where

W = weight in pounds of an individual armor unit in primary cover layer

γ_r = unit weight of armor unit in lb/cu ft (assume equals 165 lb/cu ft)

H = design wave height (H_{10}) at structure in ft

K_D = stability coefficient that varies primarily with armor unit shape, roughness, sharpness of edges, and degree of interlocking (see Table 7-8 SPM 1984)

γ = unit weight of water in lb/cu ft (assume equals 64.0 lb/cu ft for seawater)

θ = angle of structure slope relative to horizontal (assume 1:2 slope, so $\cot\theta = 2$)

The selected stability coefficient, $K_D = 1.6$, is applicable for rough angular quarry stone, an armor layer two stones thick (i.e., $n = 2$), and breaking waves at the structure head (SPM 1984). However, SPM (1984) cautions that this K_D value is unsupported by test results and should only be used for preliminary design. The New York District experience will confirm selected design stone size.

Table 7 lists the calculated stone size for each water level and H_{10} and H_b value calculated previously.

Hindcast wave data from the Wave Information Study (WIS) (Brooks and Brandon 1995) were consulted to examine nonbreaking waves and calculate stone sizes for comparison with those calculated for breaking waves. WIS Atlantic Coast Station 72 located in 88.6 ft of water offshore of the north New Jersey coast was selected. The joint occurrence of wave heights and periods were examined, and each combination of height and period observed during the 1976-1993 period was incorporated in the analysis. Using tables from SPM (1984), the deep-water unrefracted wave height was calculated for each height-period combination. Using the nearshore slope for Profile 255 and Figures 7-2 and 7-3 from SPM (1984), breaking wave height and depth were calculated for each offshore condition. Where breaking occurred offshore of the groin for each of the three water levels of interest for this study, nonbreaking wave calculations were discontinued.

Table 7
Design Stone

Period Sec	10-Year Water Level H_b ft	H_{10} ft	W tons	50-Year Water Level H_b ft	H_{10} ft	W tons	100-Year Water Level H_b ft	H_{10} Ft	W Tons
3.5	13.1	10.6	7.7	14.3	11.5	10.1	14.7	11.9	11.0
4.5	13.1	10.6	7.7	14.3	11.5	10.1	14.7	11.9	11.0
5.5	13.9	11.3	9.4	14.3	11.5	10.1	14.7	11.9	11.0
6.5	15.7	12.7	13.3	16.5	13.4	15.7	17.1	13.8	17.2
7.5	16.4	13.2	15.2	17.9	14.5	19.8	17.8	14.4	19.7
8.5	17.9	14.5	20.0	18.6	15.1	22.4	19.2	15.5	24.6
9.5	18.3	14.8	21.2	19.6	15.8	26.0	20.2	16.3	28.6
10.5	18.6	15.1	22.4	20.3	16.4	29.2	20.6	16.7	30.3
11.5	19.1	15.5	24.4	20.9	16.9	31.7	21.0	17.0	32.0
12.5	19.3	15.6	25.0	20.9	16.9	31.7	21.6	17.4	34.8

Where breaking did not occur seaward of the groin for the 17.4-, 19.0-, and 19.6-ft depths, shoaling and refraction coefficients were calculated assuming straight and parallel bottom contours and using three directions of incidence—45 deg (from the northeast), 108 deg (slightly south of due east and the direction of largest wave from WIS Station 72), and 135 deg (from the southeast). Similar to the H_{max}-to-H_{10} procedure used for breaking waves, the transformed non-breaking waves were converted to an H_{10} for use in Equation 4 along with K_D = 2.8 and $\cot\theta = 2.0$ for nonbreaking waves and structure side slopes of 1:2.

Physical model tests have been conducted to characterize rubble-mound structure damage for exposure to waves larger than the design height. These relationships can be consulted to estimate the amount of potential damage that may occur if stones smaller than that specified by the design wave are used. SPM (1984) provides a table relating wave-height ratio to percent damage. The wave-height ratio is the wave height corresponding to some damage D over the zero-damage wave height ($H_D = 0$). Percent damage is defined as the volume of armor units displaced from the structure between the center line of the structure crest width to one zero-damage wave height ($H_D = 0$) below the still-water level per unit length of structure.

Solving Equation 4 for H and forming a ratio for two wave heights result in Equation 5. To determine the damage level, let H_1 be the zero-damage (design) wave height corresponding to the zero-damage (design) weight W_1. The quantity H_2 is the smaller wave height for which zero damage occurs and gives the weight of the available stone, W_2. The H_1/H_2 ratio for specific damage levels is found in SPM (1984). Equation 5 can then be solved for W_2 given the zero-damage weight W_1 and percent damage wave-height ratio. The stone weight

corresponding to a given percent damage can then be calculated relative to the design wave and stone.

$$\frac{H_1}{H_2} = \left(\frac{W_1}{W_2}\right)^{1/3} \tag{5}$$

Figure 38 shows the percent damage potential for a range of armor weights versus breaking wave height. A separate family of curves is provided for each water level. These curves relate the damage potential of a particular stone size to water-level recurrence interval. This approach can also be expanded to estimate annual repair costs given a wave-height recurrence interval and stone-replacement cost.

Figure 39 shows the percent damage potential for a range of armor weights versus non-breaking wave height for three wave directions. Breaking and non-breaking stone size can be compared by examining the zero-percent damage curves in Figures 38 and 39.

It should be noted that the percent damage relationships from SPM (1984) were intended for situations with nonbreaking waves incident on a structure trunk. Therefore, caution is recommended where applying them in breaking waves incident on structure heads and for stone revetments.

Specification of armor weights to protect against zero damage is not practical. The designer must balance cost and availability of stone with potential damage in selecting the most appropriate armor unit stone size. The New York District has considerable experience in groin design for both the New Jersey and Long Island, New York, coasts. At Westhampton Beach on Long Island, the New York District specified armor unit weights of 10 to 14 tons for modification of the existing groin field. Existing structures at Monmouth Beach have armor weights of 4 to 10 tons. Although it is difficult to determine the extent to which these structures have been damaged, in light of the stone-sizing analysis and the greater depths at the Groin 44 extension, existing stone sizes at Monmouth Beach are likely smaller than should be used for the extension.

An armor weight (W_{50}) of 12 tons with a grade range of 10 to 14 tons is considered to be most appropriate for the Groin 44 extension. The most commonly occurring wave periods at the Long Branch wave gauge are 8.5 to 9.5 sec, which correspond to depth-limited breaking wave heights of 18, 19, and 19.5 ft for the 10-, 50-, and 100-year water levels, respectively. From Figure 38, one can see that for these water levels with an eroded profile, a structure armored with 12-ton stone will be susceptible to approximately 10, 15, and 20 percent damage, respectively. Twelve-ton stone is larger than the zero-damage stone size for any of the nonbreaking wave conditions.

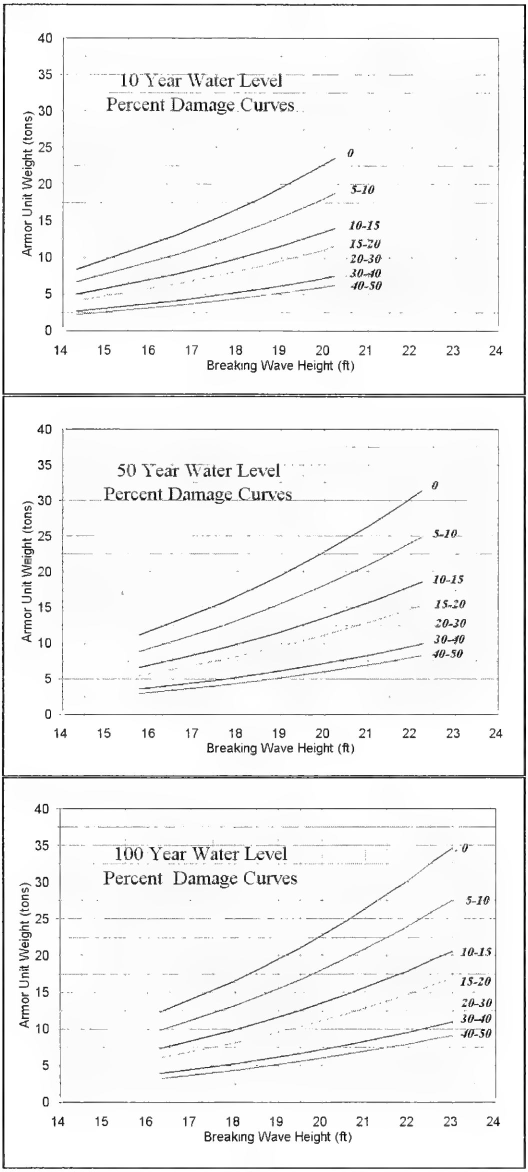


Figure 38. Percent damage curves for Groin 44 extension (breaking waves)

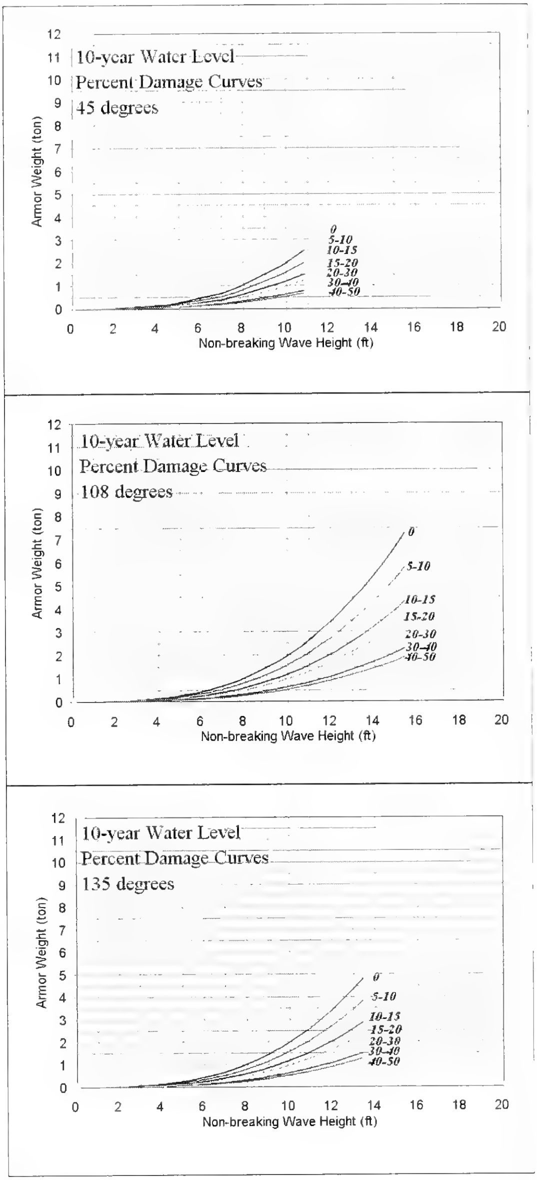


Figure 39. Percent damage curves for Groin 44 extension (nonbreaking waves (Sheet 1 of 3)

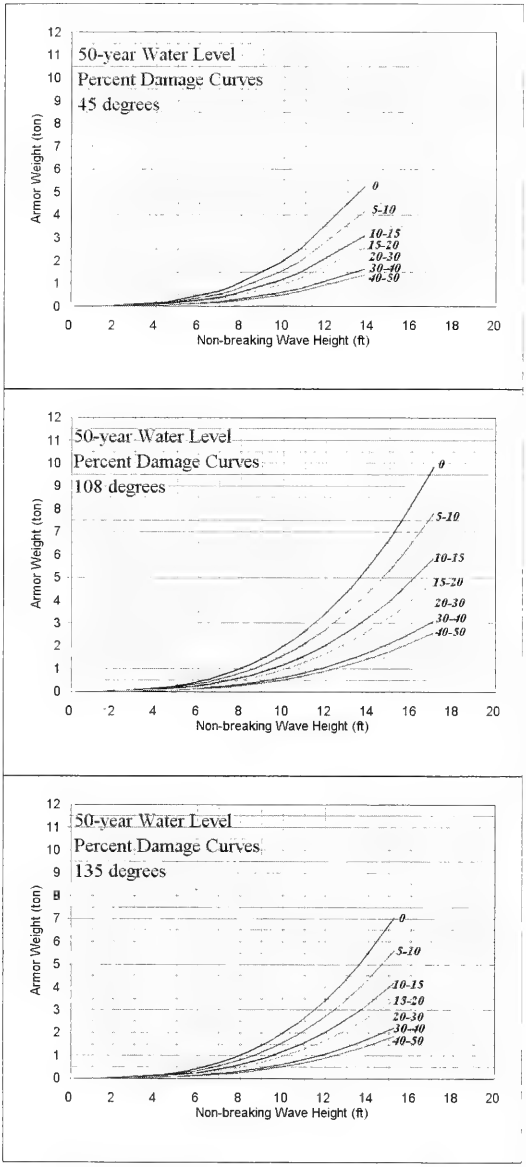


Figure 39. (Sheet 2 of 3)

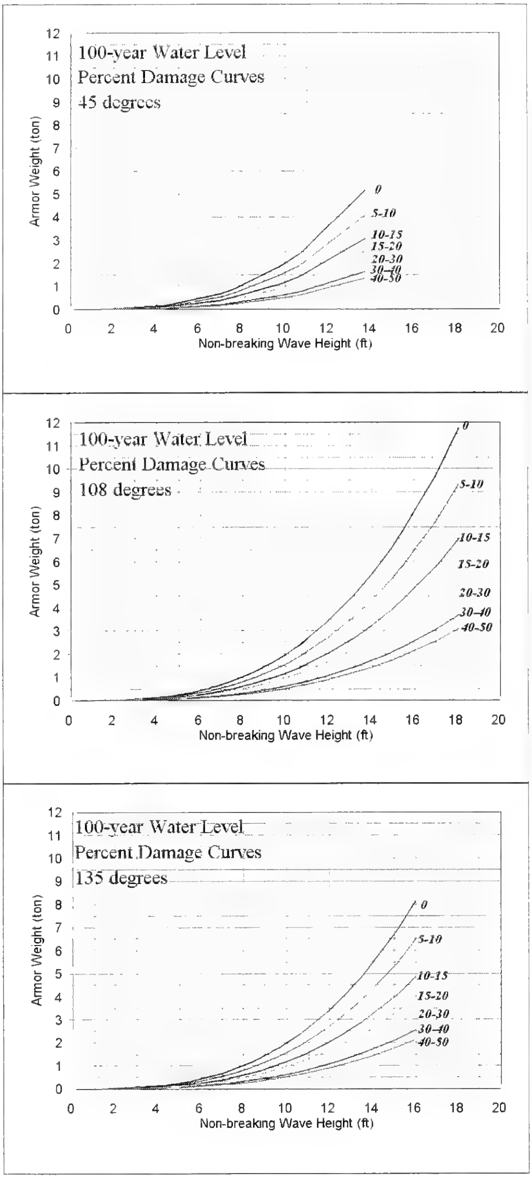


Figure 39. (Sheet 3 of 3)

Construction Details

The structure dimensions (crest width, layer thicknesses, etc.) are calculated based on equations presented in SPM (1984). Crest width calculated by Equation 6 is 15.8 ft

$$B = n\, k_{\Delta} \left(\frac{W}{\gamma_r} \right)^{1/3} \tag{6}$$

where

$B =$ crest width in ft

n = number of units in width (recommend minimum value of 3)

k_{Δ} = layer coefficient (equals 1 for 2-layer, rough quarry stone)

W = armor unit weight (12 tons)

γ_r = unit weight of armor unit in lb/cu ft (assume equals 165 lb/cu ft)

Armor layer thickness calculated by Equation 7 is 10.5 ft.

$$r = n\, k_{\Delta} \left(\frac{W}{\gamma_r} \right)^{1/3} \tag{7}$$

where

$r =$ average thickness of armor layer in ft

n = number of units in thickness (assume has a value of 2)

The number of armor units per surface area calculated by Equation 8 is 4.6 per 100 ft^2.

$$\frac{N_r}{A} = n\, k_{\Delta} \left(1 - \frac{P}{100} \right) \left(\frac{\gamma_r}{W} \right)^{2/3} \tag{8}$$

where

N_r = number of units per given surface area

A = surface area

P = porosity (equals 37 percent for 2-layer, rough quarry stone)

The primary underlayer stone size is $W/10$ or 1.2 tons (range of 1.0 to 1.4 tons), and the bedding layer ranges from $W/4000$ to $W/200$ (approximately 6 to 120 lb).

Plan and profile views of the 100-ft extension to Groin 44 are shown in Figure 40. Sections along the groin length and across the groin width are shown

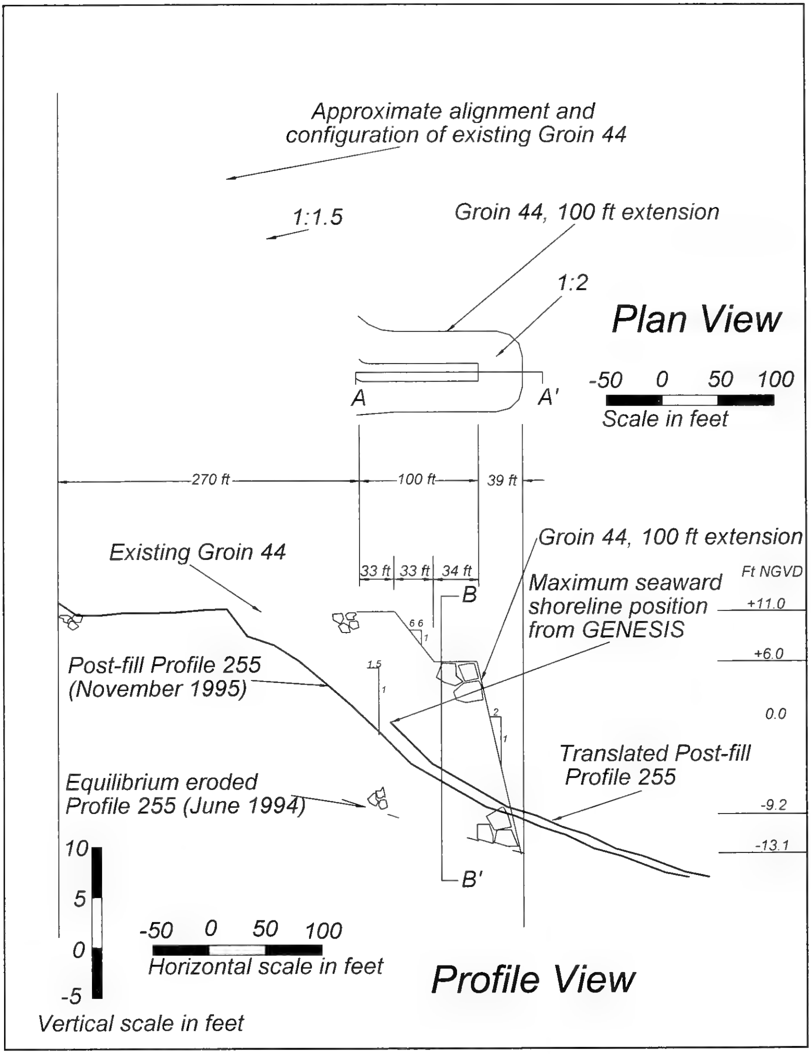


Figure 40. Plan and profile view of Groin 44 extension

in Figures 41 and 42. Note that the structure head shown in Figure 41 (outer 20 ft along the structure crest) is approximately 3 stone-sized thick with no primary underlayer.

Stone Volume/Weight Estimate

The total volume of each layer of stone was determined by reference to Figures 40, 41, and 42. The layer volume was then multiplied by the stone unit weight (165 lb/cu ft) and assuming 37 percent porosity (multiplied by 0.63). The volumes and weight of each layer are given in Table 8.

Table 8
Layer Volume and Weight of Total Stone

	Armor (W = 10 to 14 tons)	Primary Underlayer (W = 1.0 to 1.4 tons)	Bedding Layer (W = 6 to 120 lb)
Volume, cu ft	71,400	6,100	23,600
Weight, tons	3,700	320	1,200

There is a need for a tie-in to the existing structure to provide better groin performance and enhance interlocking between the existing groin and the goin extension. Quantities required for the transition sections will be determined from detailed surveys of the existing groin taken during the preparation of construction plans and specifications.

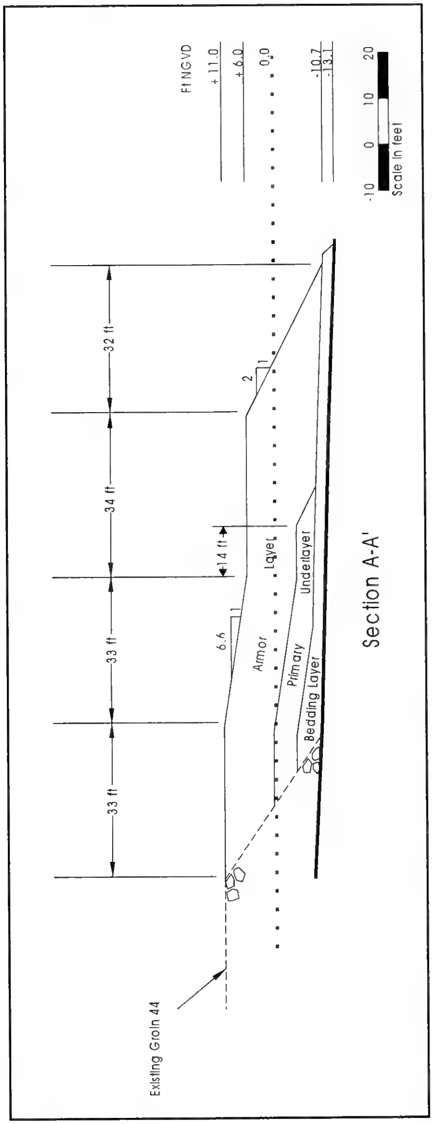


Figure 41. Section A-A' of Groin 44 extension

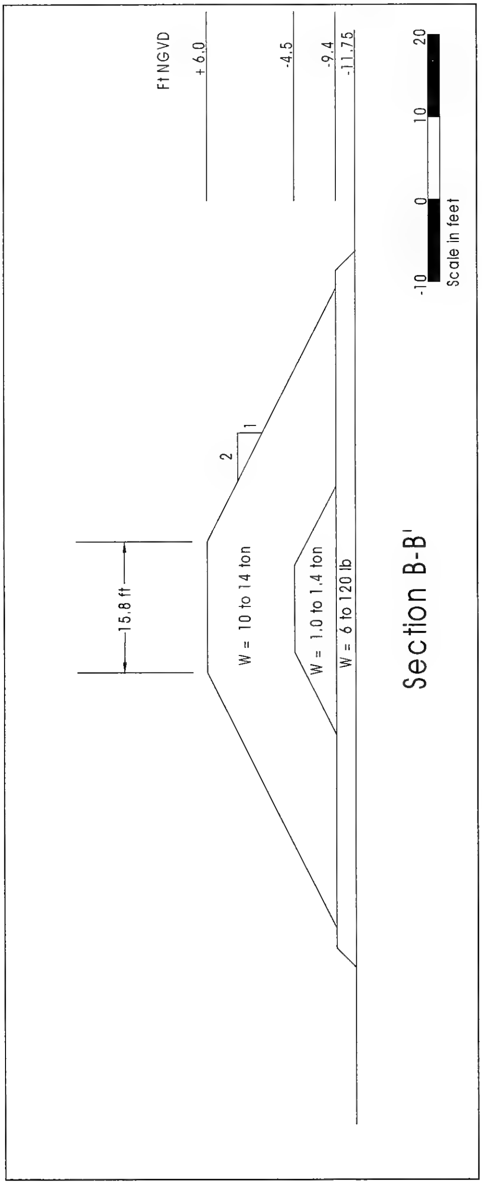


Figure 42. Section B-B' of Groin 44 extension

5 Conclusions

This chapter summarizes the two recommended alternative functional designs of five developed and evaluated in this study. The alternatives were formulated to increase the longevity of the fill placed at the Monmouth Beach erosion hot spot. A functional design considers the coastal physical processes and geometric configurations of engineering works to arrive at alternatives without giving details of the construction process and materials. Background information for the functional design is contained in Chapter 3, and Chapter 4 describes the calculations for the composition and dimensions of the groin extension associated with the recommended Alternative 3.

Functional Design

Alternative shore-protection and beach-fill retention designs were described in Chapter 3. Development of alternatives built upon previous work (CP&E 1997; Smith, Gravens, and Smith 1999) performed for the New York District. Table 2 summarizes the five alternatives evaluated in detail in this study. The alternatives were evaluated through inspection of simulation results from GENESIS and through comparison with a locally valid, empirically derived criterion for the ratio of groin length and groin spacing. Of the five alternatives evaluated, two design alternatives, Alternatives 1 and 3, are recommended for consideration by the New York District.

The selection of these two alternatives was based on GENESIS simulations, broad consideration of cost, and a criterion for minimal frequency of landward encroachment of the 75-ft protective berm at the hot spot and along the directly adjacent downdrift beach. The GENESIS model was calibrated to replicate regional longshore sand transport, thereby giving a long-term average net longshore sand transport rate. The net and gross directions of transport and the various daily to seasonal variations in transport were simulated by input of a 3-year wave record of measurements made at Long Branch.

Alternative 1 (no new structures) allows beach-fill material placed as part of the shore protection project to the south to nourish the hot spot by the natural longshore transport that is directed predominantly to the north. Simulation results based on the 3-year wave record indicate that the protective berm width within the hot spot will be less than 75 ft for approximately 60 to 80 percent of the 6-year renourishment interval. An advantage of this alternative is that

erosion of the 75-ft protective berm downdrift of Groin 44 is minimized (40-50 percent of the 6-year renourishment interval). In addition, this alternative has economic and aesthetic advantages in that no additional structures must be placed within the hot spot.

Alternative 3 (seaward extension of Groin 44 by 100 ft) impounds additional material within the hot spot and significantly reduces impacts to the protective berm. By extending Groin 44 by 100 ft, erosion of the 75-ft protective berm within the hot spot is reduced to approximately 20 percent of the 6-year renourishment interval. A disadvantage of Alternative 3 is the increased erosion downdrift of Groin 44. For a distance of approximately 3,600 ft north of Groin 44, the 75-ft protective berm experiences more episodes of landward encroachment than for Alternative 1 (Figure 21). Simulations indicated that the protective berm directly north of Groin 44 will be less than 75 ft wide approximately 40-65 percent of the 6-year renourishment interval. Cumulative berm impacts decrease with distance north of Groin 44.

The New York District may wish to consider other alternatives than Alternatives 1 and 3 as described above. For example, Alternative 2 (350-ft seaward extension of Groin 44) reduces frequency of landward encroachment of the 75-ft-wide berm to less than 5 percent, but at the expense of increasing recession frequency downdrift.

Wave conditions more energetic than the 3-year measured wave record are expected to produce impacts greater than those expressed in the design evaluations. To estimate potential variability in the recommended alternatives, a variability analysis was performed by incorporating estimated annual extremes in longshore sand transport as obtained from the 3-year wave record. The variability analysis revealed that the protective berm impacts at the hot spot are more severe for years in which the percentage of transport to the south is greater than average. Conversely, greater erosion downdrift of Groin 44 is anticipated for years in which the percentage of transport to the north is greater than average. Alternative 3 is the recommended alternative for protecting the hot spot, although Alternative 1 provides marginally fewer detrimental impacts to the area downdrift of Groin 44.

Groin Design

The structural design for the 100-ft extension of Groin 44 (Alternative 3) was developed based on consideration of the forces exerted by depth-limited breaking waves for three storm water levels. For the design, Groin 44 was extended seaward 100 ft perpendicular to shore on an eroded beach profile to produce a conservative estimate for maximum breaking wave height for a given water level. The depth-limited breaking wave height for a range of periods typical for Monmouth Beach was calculated from the depth at the toe of the structure on the beach profile and from the nearshore slope. By applying procedures outlined in SPM (1984), stone size was calculated and potential damage estimated for stone smaller than specified.

Existing structures at Monmouth Beach have a range of stone sizes (3.5 to 10.3 tons) and are generally classified as being in either fair or poor condition. Because of their condition and assuming an acceptable damage of 10 to 20 percent, an armor unit weighing 12 tons (range of 10 to 14 tons) was selected. The primary underlayer stone should be 1/10 of the armor layer weight (1.2-ton range of 1.0 to 1.4 tons) and the bedding layer 1/200 to 1/400 of the armor layer weight (6 to 120 lb).

Plan, profile, and cross-section templates (Figures 40-42, respectively) were prepared from these stone sizes and recommended layer thicknesses. Volume and weight estimates were then made for the required stone.

References

Brooks, R. M., and Brandon, W. A. (1995). "Hindcast wave information for the U.S. Atlantic Coast: Update 1976-1993 with hurricanes," Wave Information Study (WIS) Report 33, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Caldwell, J. M. (1966). "Coastal processes and beach erosion," *Journal of the Society of Civil Engineers* 53(2), 142-157. (Also reprinted with an abstract and errata as Reprint Report 1-67, U.S. Army Engineer Coastal Engineering Research Center, Corps of Engineers, 1967.)

Coastal Planning & Engineering, Inc. (CP&E). (1997). "Engineering and design for the alternative storm damage protection required for Sea Bright to Ocean Township, New Jersey, Beach Erosion Control Project, Monmouth Beach, Reach 1A," Contract Report prepared for the U.S. Army Engineer District, New York, New York.

Dean, R. G. (1977). "Equilibrium beach profiles: U.S. Atlantic and the Gulf Coasts," Department of Civil Engineering, Ocean Engineering Report No. 12, University of Delaware, Newark, DE.

__________. (1991). "Equilibrium beach profiles: Characteristics and applications," *Journal of Coastal Research* 7(1), 53-84.

Gravens, M. B., Scheffner, N. W., and Hubertz, J. M. (1989). "Coastal processes from Asbury Park to Manasquan, New Jersey," Miscellaneous Paper CERC-89-11, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

Hanson, H., Gravens, M. B., and Kraus, N. C. (1988). "Prototype applications of a generalized shoreline change numerical model." *Proceedings, 21st International Conference on Coastal Engineering*, American Society of Civil Engineers, NY, 1265-1279.

Hanson, H., and Kraus, N. C. (1989). "GENESIS: Generalized model for simulating shoreline change, Report 1: Technical reference," Technical Report CERC-89-19, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

Hanson, H., and Kraus, N. C. (1991). "Comparison of shoreline change obtained with physical and numerical models." *Proceedings, Coastal Sediments '91*, American Society of Civil Engineers, NY, 1785-1813.

Hashimoto, H., Uda, T., and Takebuchi, T. (1981). "Prediction of beach profile change in the vicinity of structures by an empirical model." *Proceedings, 28th Japanese Conference on Coastal Engineering*, Japan Society of Civil Engineers, Tokyo, 300-304.

Kraus, N. C. (1983). "Applications of a shoreline prediction model." *Proceedings, Coastal Structures '83*, American Society of Civil Engineers, NY, 632-645.

Kraus, N. C., Gravens, M., and Mark, D. J. (1988). "Coastal processes at Sea Bright to Ocean Township, New Jersey, Volume 2: Appendixes B-G," Miscellaneous Paper CERC-88-12, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

Kraus, N. C., Hanson, H., and Blomgren, S. (1994). "Modern functional design of groin systems." *Proceedings, 24th Coastal Engineering Conference*, American Society of Civil Engineers, NY, 1327-1342.

Kraus, N. C., Scheffner, N. W., Chou, L. W., Cialone, M. A., Smith, J. M., and Hardy, T. A. (1988). "Coastal processes at Sea Bright to Ocean Township, New Jersey, Volume 1: Main Text and Appendix A," Miscellaneous Paper CERC-88-12, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

Shore Protection Manual (SPM). (1984). 2nd ed., 2 Vols., U.S. Army Corps of Engineers, Coastal Engineering Research Center, U.S. Government Printing Office, Washington, DC.

Smith, S. J., Gravens, M. B., and Smith, J. M. (1999). "Monmouth Beach, New Jersey: Beach-fill 'hot spot' erosion evaluation, Report 1, Physical processes analysis," Technical Report in preparation, U.S. Army Engineer Waterways Experiment Station, Coastal and Hydraulics Laboratory, Vicksburg, MS.

U.S. Army Corps of Enginers. (1986). "Design of breakwaters and jetties," U.S. Army Corps of Engineers, Engineer Manual (EM) 1110-2-2904, Washington, DC.

U.S. Army Engineer District, New York. (1989). "Atlantic Coast of New Jersey Sandy Hook to Barnegat Inlet Beach Erosion Control Project, Section I-Sea Bright to Ocean Township, New Jersey, General Design Memorandum" Volumes I and II, U.S. Army Engineer District, New York, New York.

Appendix A Summaries of Wave Measurements at Long Branch, New Jersey, Directional Wave Gauge

This appendix presents tabular summaries of wave measurements collected at the Long Branch, New Jersey, directional wave gauge (DWG). The Long Branch DWG (40.30° N, 73.97° W) is located approximately 2.7 km south of the Monmouth Beach hot spot at a depth of approximately 10 m. The analysis of measured waves covers January 1994 through April 1998. Extended gaps in the measured wave data occur from mid-November 1994 through mid-July 1995 and October 1996 through February 1997 because of instrument failure.

Peak wave period and direction are not recorded for measured wave heights less than 0.2 m. Data recorded as "Bad Data" in the tables result from instrument error/failure or irrational recorded data. Peak wave period and direction correspond to the maximum wave energy of the measured directional spectrum. Wave directions are expressed as the azimuth (in degrees) relative to true north from which waves approach the gauge.

LONG BRANCH, NJ
LONG BRANCH, NEW JERSEY 1994 - 1998
LAT: 40.30 N, LONG: 73.97 W, DEPTH:10 M
SUMMARY OF WAVE INFORMATION BY MONTH

OCCURRENCES OF WAVE HEIGHT BY MONTH FOR ALL YEARS

Hmo(m)	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	TOTAL
0.00 - 0.49	154	234	282	231	209	222	[illegible]	356	277	341	[illegible]	[illegible]	[illegible]
0.50 - 0.99	236	169	313	367	301	[illegible]	[illegible]	[illegible]	252	230	[illegible]	[illegible]	[illegible]
1.00 - 1.49	87	93	81	91	41	34	[illegible]	[illegible]	110	78	[illegible]	41	[illegible]
1.50 - 1.99	40	24	25	23	7	17	[illegible]	[illegible]	23	26	[illegible]	[illegible]	[illegible]
2.00 - 2.49	16	12	11	6	.	7	[illegible]	3	8	8	[illegible]	[illegible]	[illegible]
2.50 - 2.99	14	7	6	2	.	.	1	.	1	2	[illegible]	1	[illegible]
3.00 - 3.49	8	9	7	.	.	.	.	.	2	2	[illegible]	[illegible]	[illegible]
3.50 - 3.99	2	4	4	.	.	.	.	.	1	1	[illegible]	.	[illegible]
4.00 - 4.49	.	.	.	.	.	.	.	.	.	.	.	.	[illegible]
BAD DATA	371	[illegible]	201	180	186	182	121	.	46	17	[illegible]	[illegible]	[illegible]
TOTAL	928	849	930	900	744	720	744	744	720	705	[illegible]	744	[illegible]

OCCURRENCES OF PEAK PERIOD BY MONTH FOR ALL YEARS

Tp(sec)	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	TOTAL
[illegible]	9	16	26	35	22	15	16	18	36	54	[illegible]	[illegible]	[illegible]
4.0 - 4.9	37	32	41	58	39	48	36	46	46	45	[illegible]	[illegible]	[illegible]
5.0 - 5.9	33	[illegible]	47	74	34	37	45	40	57	35	23	22	[illegible]
6.0 - 6.9	43	33	57	68	80	69	110	154	50	37	[illegible]	21	[illegible]
7.0 - 7.9	40	43	72	96	93	108	115	151	65	51	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	128	168	153	154	163	198	102	114	[illegible]	[illegible]	[illegible]
[illegible]	135	139	190	132	81	86	97	77	110	155	[illegible]	[illegible]	[illegible]
[illegible]	74	61	64	48	15	6	14	[illegible]	48	48	[illegible]	[illegible]	[illegible]
[illegible]	68	38	47	23	21	9	15	[illegible]	51	39	[illegible]	13	[illegible]
12.0 - 12.9	20	21	19	5	7	4	8	14	44	39	20	23	224
13.0 - 13.9	.	.	.	.	.	.	.	.	.	.	.	.	0
[illegible] - 14.9	4	16	4	.	.	2	3	16	41	15	[illegible]	[illegible]	[illegible]
[illegible]	.	.	.	.	.	.	.	.	.	.	.	.	[illegible]
[illegible]	.	3	.	.	.	.	1	[illegible]	9	3	.	.	[illegible]
17.0 - 17.9	.	.	.	.	.	.	.	.	.	.	.	.	[illegible]
18.0 - 18.9	.	.	.	.	.	.	.	.	1	.	.	.	1
19.0 - 19.9	.	.	.	.	.	.	.	.	.	.	.	.	0
20.0 - LONGER	.	.	.	.	.	.	.	.	.	.	.	.	0
BAD DATA	401	325	235	193	199	182	121	.	60	40	[illegible]	446	[illegible]
TOTAL	928	849	930	900	744	720	744	[illegible]	720	705	[illegible]	[illegible]	9454

OCCURRENCES OF PEAK DIRECTION BY MONTH FOR ALL YEARS

Dp(deg)	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	TOTAL
DIRECTION BAND & CENTER													
348.75 - [illegible]	.	.	.	.	.	.	.	.	.	.	[illegible]	.	[illegible]
[illegible] - 33.74 (22.[illegible]	.	[illegible]	[illegible]	[illegible]	[illegible]	.	.	.	[illegible]	[illegible]	3	[illegible]	36
33.[illegible] - 56.24 (45.[illegible]	[illegible]	4	3	[illegible]	[illegible]	[illegible]	1	.	8	9	[illegible]	4	46
56.[illegible] - [illegible] 67.5	[illegible]	[illegible]	8	3	[illegible]	.	[illegible]	8	9	[illegible]	[illegible]	7	66
78.[illegible] - 101.24 (90.[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1360
101.[illegible] - 123.74 (112.[illegible]	[illegible]	134	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	304	298	[illegible]	151	[illegible]	2335
123.[illegible] - 146.24 (135.[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	241	[illegible]	[illegible]	265	164	146	[illegible]	[illegible]	[illegible]
146.25 - [illegible].74 (157.[illegible]	[illegible]	[illegible]	49	[illegible]	[illegible]	[illegible]	[illegible]	49	36	[illegible]	[illegible]	[illegible]	562
168.[illegible] - [illegible]	.	.	3	[illegible]	[illegible]	4	1	3	1	[illegible]	6	[illegible]	[illegible]
[illegible] - 213.74 (202.5)	.	.	.	.	.	.	.	.	.	.	.	.	0
213.[illegible] 236.24 (225.[illegible]	.	.	.	.	.	.	.	.	.	.	.	.	0
236.[illegible] - 258.74 (247.5	.	.	.	.	.	.	.	.	.	.	.	.	0
[illegible] - 281.24 (270.[illegible]	.	.	.	.	.	.	.	.	.	.	.	.	0
281.[illegible] - 303.74 (292.[illegible]	.	.	.	.	.	.	.	.	.	.	.	.	0
303.[illegible] - 326.24 (315.[illegible]	.	.	.	.	.	.	.	.	.	1	.	.	1
326.25 - [illegible].74 (337.[illegible]	.	.	.	.	.	.	.	.	.	.	.	.	0
BAD DATA	562	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	121	.	65	[illegible]	[illegible]	446	[illegible]
TOTAL	[illegible]	[illegible]	930	[illegible]	744	[illegible]	[illegible]	744	720	705	[illegible]	744	9454

OCCURRENCES OF WAVE HEIGHT AND PEAK PERIOD FOR 45-DEG DIRECTION BANDS

(337.50 - 22.49) 0.0 DEG

Tp(sec)

Hmo(m)	3.0-4.9	5.0-6.9	7.0-8.9	9.0-10.9	11.0-12.9	13.0-14.9	15.0-16.9	17.0-18.9	19.0-20.9	21.0-LONGER	TOTAL
0.00 - 0.99	9	.	.	.	.	.	.	.	.	.	9
1.00 - 1.99	.	.	.	.	.	.	.	.	.	.	0
2.00 - 2.99	.	.	.	.	.	.	.	.	.	.	0
3.00 - 3.99	.	.	.	.	.	.	.	.	.	.	0
4.00 - 4.99	.	.	.	.	.	.	.	.	.	.	0
5.00 - 5.99	.	.	.	.	.	.	.	.	.	.	0
TOTAL	9	0	0	0	0	0	0	0	0	0	9

(22.50 - 67.49) 45.0 DEG

Tp(sec)

Hmo(m)	3.0-4.9	5.0-6.9	7.0-8.9	9.0-10.9	11.0-12.9	13.0-14.9	15.0-16.9	17.0-18.9	19.0-20.9	21.0-LONGER	TOTAL
0.00 - 0.99	88	.	.	.	.	1	.	.	.	.	89
1.00 - 1.99	6	.	.	.	.	.	.	.	.	.	6
2.00 - 2.99	.	.	.	.	.	.	.	.	.	.	0
3.00 - 3.99	.	.	.	.	.	.	.	.	.	.	0
4.00 - 4.99	.	.	.	.	.	.	.	.	.	.	0
5.00 - 5.99	.	.	.	.	.	.	.	.	.	.	0
TOTAL	94	0	0	0	0	1	0	0	0	0	95

(67.50 - 112.49) 90.0 DEG

Tp(sec)

Hmo(m)	3.0-4.9	5.0-6.9	7.0-8.9	9.0-10.9	11.0-12.9	13.0-14.9	15.0-16.9	17.0-18.9	19.0-20.9	21.0-LONGER	TOTAL
0.00 - 0.99	174	165	413	609	352	64	6	.	.	.	1783
1.00 - 1.99	32	162	166	148	52	5	1	.	.	.	566
2.00 - 2.99	.	10	59	30	7	.	.	.	.	.	106
3.00 - 3.99	.	.	15	21	6	.	.	.	.	.	42
4.00 - 4.99	.	.	.	.	.	.	.	.	.	.	0
5.00 - 5.99	.	.	.	.	.	.	.	.	.	.	0
TOTAL	206	337	653	808	417	69	7	0	0	0	2497

112.50 - 157.49, 135.0 DEG

Tp(sec)

Hmo(m)	3.0-4.9	5.0-6.9	7.0-8.9	9.0-10.9	11.0-12.9	13.0-14.9	15.0-16.9	17.0-18.9	19.0-20.9	21.0-LONGER	TOTAL
0.00 - 0.99	228	677	1470	776	107	32	8	1	.	.	3299
1.00 - 1.99	12	98	128	105	35	16	8	.	.	.	402
2.00 - 2.99	.	2	16	1	.	.	.	.	.	.	19
3.00 - 3.99	.	.	.	2	.	.	.	.	.	.	2
4.00 - 4.99	.	.	.	.	.	.	.	.	.	.	0
5.00 - 5.99	.	.	.	.	.	.	.	.	.	.	0
TOTAL	240	777	1614	884	142	48	16	1	0	0	3722

(157.50 - 202.49) 180.0 DEG

Tp(sec)

Hmo(m)	3.0-4.9	5.0-6.9	7.0-8.9	9.0-10.9	11.0-12.9	13.0-14.9	15.0-16.9	17.0-18.9	19.0-20.9	21.0-LONGER	TOTAL
0.00 - 0.99	205	54	2	.	.	.	.	.	.	.	261
1.00 - 1.99	1	.	.	.	.	.	.	.	.	.	1
2.00 - 2.99	.	.	.	.	.	.	.	.	.	.	0
3.00 - 3.99	.	.	.	.	.	.	.	.	.	.	0
4.00 - 4.99	.	.	.	.	.	.	.	.	.	.	0
5.00 - 5.99	.	.	.	.	.	.	.	.	.	.	0
TOTAL	206	54	2	0	0	0	0	0	0	0	262

([illegible] - 247.49) 225.0 DEG

Hmo (m)	3.0- 4.9	5.0- 6.9	7.0- 8.9	9.0- 10.9	11.0- 12.9	13.0- 14.9	15.0- 16.9	17.0- 18.9	19.0- 20.9	21.0- LONGER	TOTAL
[illegible] - [illegible]	.	.	.	.	.	.	.	.	.	.	[illegible]
[illegible] - 1.99	.	.	.	.	.	.	.	.	.	.	[illegible]
[illegible] - [illegible]	.	.	.	.	.	.	.	.	.	.	[illegible]
3.00 - 3.99	.	.	.	.	.	.	.	.	.	.	[illegible]
4.00 - 4.99	.	.	.	.	.	.	.	.	.	.	[illegible]
[illegible] - [illegible]	.	.	.	.	.	.	.	.	.	.	[illegible]
TOTAL	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

(247.50 - [illegible]) 270.0 DEG

Hmo (m)	3.0- 4.9	5.0- 6.9	7.0- 8.9	9.0- 10.9	11.0- 12.9	13.0- 14.9	15.0- 16.9	17.0- 18.9	19.0- 20.9	21.0- LONGER	TOTAL
[illegible] - [illegible]	.	.	.	.	.	.	.	.	.	.	[illegible]
1.00 - 1.99	.	.	.	.	.	.	.	.	.	.	[illegible]
2.00 - 2.99	.	.	.	.	.	.	.	.	.	.	[illegible]
3.00 - 3.99	.	.	.	.	.	.	.	.	.	.	[illegible]
4.00 - 4.99	.	.	.	.	.	.	.	.	.	.	[illegible]
5.00 - 5.99	.	.	.	.	.	.	.	.	.	.	[illegible]
TOTAL	[illegible]	[illegible]	[illegible]	[illegible]	0	0	0	0	0	0	0

(292.50 - 337.49) 315.0 DEG

Hmo (m)	3.0- 4.9	5.0- 6.9	7.0- 8.9	9.0- 10.9	11.0- 12.9	13.0- 14.9	15.0- 16.9	17.0- 18.9	19.0- 20.9	21.0- LONGER	TOTAL
0.00 - 0.99	.	.	.	.	1	.	.	.	.	.	1
1.00 - 1.99	.	.	.	.	.	.	.	.	.	.	0
2.00 - 2.99	.	.	.	.	.	.	.	.	.	.	0
3.00 - 3.99	.	.	.	.	.	.	.	.	.	.	0
[illegible] - [illegible]	.	.	.	.	.	.	.	.	.	.	0
[illegible] - [illegible]	.	.	.	.	.	.	.	.	.	.	0
TOTAL	[illegible]	[illegible]	[illegible]	[illegible]	1	0	0	0	0	0	1

ALL DIRECTIONS

Hmo (m)	3.0- 4.9	5.0- 6.9	7.0- 8.9	9.0- 10.9	11.0- 12.9	13.0- 14.9	15.0- 16.9	17.0- 18.9	19.0- 20.9	21.0- LONGER	BAD DATA	TOTAL
0.00 - 0.99	[illegible]	[illegible]	[illegible]	[illegible]	477	97	14	1	.		262	[illegible]
1.00 - 1.99	[illegible]	[illegible]	[illegible]	[illegible]	95	21	9	.	.		,	1057
2.00 - 2.99	.	13	[illegible]	[illegible]	[illegible]	.	.	.	.		,	140
3.00 - 3.99	.	.	[illegible]	[illegible]	[illegible]	.	.	.	.		,	50
4.00 - 4.99	.	.	.	.	.	.	.	.	.		,	[illegible]
5.00 - 5.99	.	.	.	.	.	.	.	.	.		,	
BAD DATA	.	.	.	.	.	.	.	.	.		[illegible]	[illegible]
TOTAL	785	[illegible]	2367	[illegible]	587	118	23	1	0	0	[illegible]	[illegible]

SUMMARY OF MEAN Hmo(m) BY MONTH AND YEAR

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	MEAN
1994	0.72	0.66	0.69	0.62	0.63	0.55	0.58	0.57	0.56	0.51	0.67	0.00	0.62
1995	0.00	0.00	0.00	0.00	0.00	0.00	0.60	0.85	0.85	0.67	0.72	0.46	0.70
1996	0.92	0.61	0.71	0.76	0.63	0.56	0.59	0.52	0.79	0.84	0.89	0.00	0.69
1997	0.00	0.65	0.65	0.72	0.60	0.75	0.59	0.58	0.59	0.65	0.87	0.64	0.66
1998	1.01	1.17	0.84	0.76	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.94
MEAN	0.89	0.79	0.72	0.71	0.62	0.63	0.59	0.63	0.70	0.65	0.77	0.55	

MAX Hmo(m)*10 WITH ASSOCIATED Tp(sec) AND Dp(deg/10) BY MONTH AND YEAR

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	MAX
1994	3810**	31 9**	4112**	13 510	17 812	15 914	11 715	19 810	36 911	21 711	24 911	0 0 0	4112**
1995	0 0 0	0 0 0	0 0 0	0 0 0	0 0 0	0 0 0	10 813	19 710	18 713	20 912	421110	25 910	421110
1996	391210	17 6 6	38 9 9	27 811	16 911	141012	27 813	17 610	31 810	37 910	17 7**	0 0 0	391210
1997	0 0 0	13 913	22 710	25 9 9	18 714	26 911	26 710	31 813	24 7**	211010	351010	331010	351010
1998	36 910	3910 9	381010	26 810	0 0 0	0 0 0	0 0 0	0 0 0	0 0 0	0 0 0	0 0 0	0 0 0	3910 9
MAX	391210	3910 9	4112**	27 811	18 714	26 911	27 813	31 813	36 911	37 910	421110	331010	

MAX Hmo(m): 4.2 MAX Tp(sec): 11. MAX Dp(deg): 98. DATE(gmt): 95111501

REPORT DOCUMENTATION PAGE

Form Approved
OMB No. 0704-0188

1. AGENCY USE ONLY *(Leave blank)*

2. REPORT DATE
July 1999

3. REPORT TYPE AND DATES COVERED
Report 2 of a series

4. TITLE AND SUBTITLE
Monmouth Beach, New Jersey: Beach-Fill "Hot Spot" Erosion Evaluation; Report 2, Functional Design of Shore-Protection Alternatives for Beach-Fill Longevity

5. FUNDING NUMBERS

6. AUTHOR(S)
S. Jarrell Smith, Gregory L. Williams, Nicholas C. Kraus

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)
U.S. Army Engineer Waterways Experiment Station
3909 Halls Ferry Road, Vicksburg, MS 39180-6199

8. PERFORMING ORGANIZATION REPORT NUMBER
Technical Report CHL-99-13

9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)
U.S. Army Engineer District, New York
26 Federal Building
New York, NY 10278-0090

10. SPONSORING/MONITORING AGENCY REPORT NUMBER

11. SUPPLEMENTARY NOTES
Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.

12a. DISTRIBUTION/AVAILABILITY STATEMENT
Approved for public release; distribution is unlimited.

12b. DISTRIBUTION CODE

13. ABSTRACT *(Maximum 200 words)*

The U.S. Army Engineer District, New York, is constructing Section I-Sea Bright to Ocean Township, New Jersey, of the Atlantic Coast of New Jersey — Sandy Hook to Barnegat Inlet Beach Erosion Control Project. Within the initial portion of this project, a zone of accelerated shoreline erosion has developed near the southern boundary of Monmouth Beach, New Jersey. This report investigates shore-protection measures for the eroding beach with the goal of maintaining a 23-m-wide protective berm over the 6-year renourishment interval. The report evaluates performance of a nearby groin field to establish groin design criteria, proposes and investigates the merits of five design alternatives employing a frequency-of-violation criterion and the shoreline change model GENESIS, and recommends a functional design to reduce the high rate of beach erosion.

14. SUBJECT TERMS

Beach fill	Longshore sediment transport
Erosion	Monmouth Beach, New Jersey
Groins	Numerical simulation

15. NUMBER OF PAGES
77

16. PRICE CODE

17. SECURITY CLASSIFICATION OF REPORT
UNCLASSIFIED

18. SECURITY CLASSIFICATION OF THIS PAGE
UNCLASSIFIED

19. SECURITY CLASSIFICATION OF ABSTRACT

20. LIMITATION OF ABSTRACT

NSN 7540-01-280-5500

Standard Form 298 (Rev. 2-89)
Prescribed by ANSI Std. Z39-18
298-102

DEPARTMENT OF THE ARMY

ENGINEER RESEARCH AND DEVELOPMENT CENTER, CORPS OF ENGINEERS
WATERWAYS EXPERIMENT STATION, 3909 HALLS FERRY ROAD
VICKSBURG, MISSISSIPPI 39180-6199

Official Business

VICKSBURG MS AUG 17'99

US POSTAGE 01.13 H METER 576815

SPECIAL
FOURTH CLASS
BOOKS/FILM

256/L25/ 1
DATA/DOCUMENT LIBRARY, WHOI
MCLEAN LAB, MS #8
360 WOOD HOLE ROAD
WOODS HOLE MA 02543-1539